Railway subgrade behaviour under flood conditions: an experimental study at full-scale

By

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Abstract

Railway track is constructed on compacted soil which is characteristically unsaturated, however, the analysis and design are not to be considered based on unsaturated soil mechanics. The main objectives of this research are to investigate track behaviour, particularly, subgrade behaviour in the event of both flooding and during the recovery period. The investigation includes hydro-mechanical behaviour and the cyclic wetting and drying effect on subgrade. Previous research has been carried out on superstructure but little information is available regarding subgrade behaviour in different conditions (saturated and unsaturated). Inadequate drainage design or blockage of existing drainage can cause substantial damage to rail track. Furthermore, the wetting and drying cycles, due to frequent flooding, changes the soil behaviour significantly; therefore, it is essential to have an understanding of subgrade behaviour in both unsaturated and saturated conditions.

The design and evaluation of subgrade behaviour is primarily based on construction loading. Environmental changes and the impact on subgrade behaviour have rarely been considered during design and maintenance work. Railway track experiences cyclic wetting and drying due to seasonal variations which significantly influence track performance. To improve both short and long-term performance with minimal maintenance, it is essential to consider environmental changes during design and assessment of subgrade behaviour; especially, for the extreme case of flooding.

A series of full-scale experiments was performed to investigate track performance during and after flooding and the subsequent recovery period. Soil suction, which is a key parameter in unsaturated soil, was measured using the filter-paper technique. Subgrade stiffness was measured by the plate load test. A relationship was established between subgrade modulus, moisture content and matric suction. In order to investigate the effect of cyclic wetting and drying, the pressure-plate and filter-paper tests were conducted to obtain the water retention curve. A track settlement model was proposed based on soil suction hysteresis. The model can predict track settlement based on whether, the track is following wetting or drying path. Soil suction has a significant influence on subgrade behaviour. This research, therefore, highlights the importance of taking into account suction hysteresis in the design of railway track and the assessment of maintenance work required. The results showed a novel way to investigate and assess subgrade behaviour.

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List of Acronyms and Symbols

d	Particle size or distance between two flat platens
d_0	Reference particle size
E	Young's modulus
Es	Secant modulus
Et	Tangent modulus at zero axial strain
F	Force
Mr	Resilient modulus
ν	Poisson's ratio
p d	Particle size mm
D_{10}	Particle size for which 10% is finer (effective size) (mm)
D_{30}	Particle size for which 30% is finer mm
D_{50}	Mean particle size (Particle size for which 50% is finer) (mm)
D_{60}	Particle size for which 60% is finer mm
e	Void ratio
e_0	Initial void ratio
ei	Void ratio at natural water content
e_f	Void ratio at saturation
i_c	Collapse Potential
R	Universal gas constant Jmol-1K-1
R_h	Relative humidity (100P / P)
R_s	Radius of the meniscus and liquid bridge respectively m
Sr	Degree of saturation %
t	Relative temperature measured on the Celsius scale
T	Absolute temperature
Ts	Surface tension
u_a	Pore air pressure (kPa)
u_w	Pore water pressure (kPa)
Δu	Pressure difference across the air-water interface (kPa)
V	Volume
w	Gravimetric water content (%)
w_{opt}	Optimum moisture content (%)
w_r	Residual gravimetric water content (%)

w_s	Saturated gravimetric water content (%)
π	osmotic pressure.
σ	total stress.
σ'	effective stress ($\sigma - u_w$).
ψ	Total suction (kPa)
ψ_m	Matric suction (kPa)
ψ_a	Air-entry value (kPa)
ψ_r	Residual value (kPa)
ψ_w	Water-entry value (kPa)
χ	Soil parameter related to the degree of saturation.

List of Publications

Hasnayn, M. M., Medero, G. M. & Woodward, P. K. (2015). Railway track performance during and post flooding. *In:* WINTER, M., SMITH, D., ELDRED, P. & TOLL, D. (eds.) *Infrastructure.* 2. ICE Publishing. ISBN: 9780727760678.

Valdes-Abellan, J., Candela, L., Medero, G.M., Buckman J., and **Hasnayn, M.M**. (2015), "ESEM results and changes in wettability patterns within soil: three years irrigation with slightly-salted water." *European Geosciences Union General Assembly 2015*, Vienna, Austria, 12 – 17 April 2015

Hasnayn, M.M., Medero, G.M. and Woodward, P.K. (2015), "Investigation of flooding effect on railway track performance." International Journal of Railway Technology **4** (2).

Hasnayn, M.M., Medero, G.M. and Woodward, P.K. (2013a), "Small scale investigation of collapse potential of railway track." *Proceedings of the* 1st *Postgraduate Conference Infrastructure and Environment Scotland*, Heriot-Watt University, Edinburgh, UK.

Hasnayn, M.M, Medero, G.M. and Woodward, P.K. (2013b), "A full scale investigation of railway track performance under flooding condition." *Proceedings of the 12th International Railway Engineering Conference,* London, UK.

Hasnayn, M.M, Medero, G.M. and Woodward, P.K. (2013c), "A full-scale experiments and investigation of effect of flooding on track performance." '*Proceedings of the 1st Railway Track Science & Engineering Workshop 'Ballast: Issues and Challenges'*, Paris, France

CHAPTER ONE- INTRODUCTION

1.1 Background

Railway track performance and maintenance depends on substructure behaviour and support; particularly, the underlying subgrade soil. The railway track experiences cyclic wetting and drying due to seasonal variations. Previous research has focused on superstructure behaviour because these are easily accessible and maintainable (Priest and Powrie, 2009). Little research has been undertaken on substructure; particularly subgrade soil behaviour under flooding conditions or after a flooding event. Despite clear evidence, the impact of flooding on railway track foundation has rarely been investigated. This current research focusses on an investigation including subgrade soil softening, the regain of subgrade strength after drying and possible action to avert subgrade damage during the recovery period.

Compacted subgrade soil is usually in an unsaturated condition and it is important to remain in this state if it is to provide optimum service (Mancuso et al., 2002; Yang et al., 2008; Siekmeier, 2011). It is necessary, therefore, to understand unsaturated soil behaviour to ensure a safe, low maintenance and economical design. Generally, unsaturated soil can either be natural unsaturated soil or compacted soil. The latter is frequently used in engineering applications involving dynamic or cyclic loading; for example, in rail beds, pavements and machine foundations (Khosravi and McCartney, 2009). In conventional geotechnics, soil is often considered as being either completely dry or fully saturated (Fredlund and Rahardjo, 1993; Fredlund et al., 1996; Uchaipichat, 2010). Moisture content is an important factor which governs subgrade performance which will depend on both climate and traffic loading. A change in soil moisture content will influence a number of soil properties such as degree of saturation, void ratio, suction, permeability, dry density and shear strength. The subgrade moisture content varies with climatic condition; in particular, during rainy periods an increase in moisture content can lead to subgrade distress or shear failure under cyclic loading. However, only the moisture content cannot describe appropriately the behaviour of subgrade soil (unsaturated). The behaviour of unsaturated soil is influenced by the presence of air and water in the pore space, which influences the stress state by air and water pressure (Mancuso et al., 2002). Unsaturated soils have been avoided due to their complicated behaviour and at present, no simple theory which adequately describes the engineering behaviour of unsaturated soil (Fredlund and Rahardjo, 1993; Fredlund et al., 1996; Fredlund, 2000; Atkinson, 2007; Kodikara, 2012).

Subgrade soils are initially in an unsaturated state, however, over time they may fluctuate between a saturated and unsaturated condition. Therefore, it is essential to characterise the hydraulic and mechanical responses under different soil suction conditions, including the saturated state (Vinale et al., 1999). In spite of clear evidence, the behaviour of soils which are used in construction has rarely been investigated under controlled-suction conditions (Rampino et al., 1999; Vinale et al., 1999; Mancuso et al., 2002). However, over the past few decades some work has been undertaken in terms of unsaturated soil; particularly, geotechnical analysis and design. Many examples in geotechnical engineering practice are problems of partial saturation. For example, slope stability which depends on the state of saturation. The natural slope or man-made slope (i.e. embankment) shows a better resistance during the drying phase. In the wetting phase, the pore water pressure increased resulting in slope failure.

Both the short and long term performance of the subgrade are dependent on environmental conditions. The behaviour of soil changes with environmental changes; therefore, soils in the real life experience various wetting and drying cycles, thus various suction histories (Ng and Xu, 2012). Subgrade design is generally based on construction loading but, for the subgrade's long-term performance environmental changes also need to be considered; because the soil behaviour changes due to hysteresis associated with changes of soil suction (Dawson and Correia, 1996; Frost et al., 2004). Climate change is impacting on railway substructure and embankments; for example, a prolonged dry period can trigger embankment shrinkage which leads to deformation of the rail-track, whereas, wet periods reduce the shear strength of soil, so causing embankment failure (Wilks, 2010).

The substructure of railway track is primarily focussed on the ballast and correction of track geometry, with subgrade as a second priority. Selig and Cantrell (2001) reported that the cost of maintenance and deterioration of track components are directly associated with drainage or subgrade conditions. It was observed by Ghataora and Rushton (2012) that the subgrade soil affected, most particularly, the upper subgrade surface layer under cyclic loading and the presence of water. Brough et al. (2006) stated that the global track stiffness is dependent on the subgrade thus the deterioration of vertical track geometry.

In February 2014, the Windsor to London railway track was flooded; with the water was coming through the ground; the location was approximately 200m from the river Thames (Westcott, 2014) as shown in Figure 1.1.

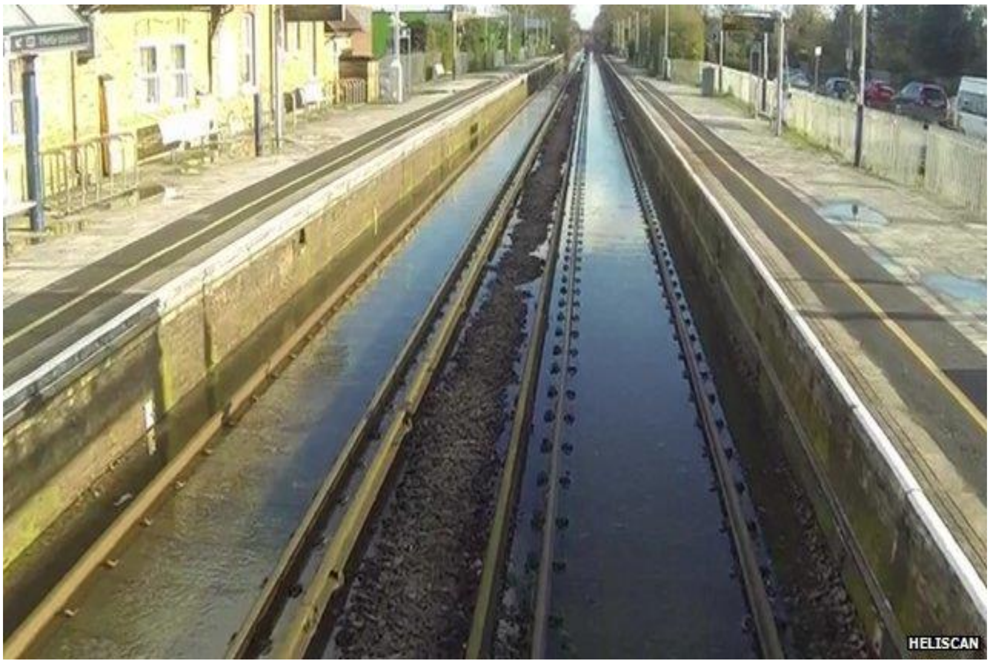


Figure 1.1 Flooded track due to saturated ground near the Thames in Berkshire (Westcott, 2014)

Another example of flooded track is presented in Figure 1.2 located near Dalton, North Yorkshire.

Figure 1.2 Flooded track near Dalton North Yorkshire (Allen, 2012)

This research undertakes an experimental study to investigate and understand the behaviour of subgrade in terms of both saturated and unsaturated conditions and how this behaviour changes with water content associated with soil suction. In particular, the project focuses on the impact of flooding on railway structure and the subsequent drying or recovery period. Cyclic wetting and drying effect was also studied. To understand track behaviour the Geopavement and Railway Accelerated Fatigue Testing (GRAFT) facility at Heriot Watt University was used for this investigation; this facility allows full-scale testing. The filter paper and pressure plate techniques were used to obtain the soil suction and water retention curves, whereas the volumetric behaviour and collapse potential was investigated by the oedometer test.

1.2 Research aim, objectives and significance of the study

The main aim of this research was to investigate subgrade soil behaviour during and after flooding. The following specific objectives were set for this research investigation:

- To undertake full-scale testing during conditions of flood and the influence of sand blanketing on track performance.
- To study the subgrade behaviour in the post flooding (recovery) period.

- To investigate the influence of moisture content, soil suction and track stiffness (hence track performance) and develop a relationship between soil suction (matric), moisture content and subgrade modulus.
- To investigate and understanding the cyclic wetting and drying (hence suction hysteresis) effect on subgrade behaviour and develop a track settlement model based on wetting and drying path.

The outcome of this research will allow an understanding of subgrade soil behaviour and track performance, particularly during and after flooding event. The results will help in the geotechnical design of the subgrade and assessment of maintenance work. The study will also help in an understanding the influence of soil suction on track performance and in the development of guidelines to analysis of subgrade behaviour. This investigation shows the importance of subgrade soil behaviour which has been neglected over a long period of time. This research has attempted to study and analyse the behaviour of subgrade soil by using unsaturated soil mechanics, an area which has, hitherto, not been used in track design.

1.3 Thesis outline

This thesis is arranged into seven Chapters. In each chapter, a relevant literature review is presented; however, a comprehensive overview is given in chapter 2. An outline of each chapter is presented below:

Chapter 1 contributes a general introduction to the research and states the research objectives.

Chapter 2 presents a literature review consisting of two parts: the first part briefly describes track behaviour and subgrade related problems; the second part focuses on the behaviour of unsaturated soil mechanics.

Chapter 3 describes the studied material, characteristics and experimental programme are conducted in this research.

Chapter 4 presents the results and analysis of experiment one at initial dry condition, after flooding at wet condition and the during recovery period. This chapter describes the impact of flooding on track behaviour.

In Chapter 5, analysis of the influence of traditional sand blanketing is offered. A sand blanket was put in place and the track was flooded for a second time. A second test was conducted without drained water from the track and a sand blanket to investigate the track performance, if drainage were to be blocked in real life. A final test was performed with a new subgrade surface layer.

Chapter 6 explains subgrade soil behaviour from the perspective of unsaturated soil mechanics. The investigations include the influence of cyclic wetting and drying and relationships between suction, stiffness and collapse behaviour.

Chapter 7 summarises the research. Recommendations for the future work are given in this chapter.

CHAPTER TWO- LITERATURE REVIEW

2.1 Introduction

This chapter focuses on a review of literature on railway subgrade and hydro-mechanical behaviour of unsaturated soil. It is divided in two parts: the first part reviews railway subgrade behaviour and associated problems. The second part reviews the unsaturated soil behaviour and suction measurement techniques. The influence of cyclic wetting and drying on mechanical behaviour of unsaturated soil is also discussed.

2.2 Railway substructure

The performance of a railway track depends on the behaviour of subgrade materials. The demand for faster and heavier train services raises the need for good support from the track's underlying substructure. Subgrade is the main factor controlling track stiffness and also protecting the vertical track geometry from settlement (Brough et al., 2006). Generally, ballasted railway track is divided in two zones viz. the superstructure and the substructure. The superstructure comprises rails, sleepers and fastening system and the substructure consists of the ballast, sub-ballast and subgrade. Figure 2.1 shows the different components of a railway track's superstructure and substructure. The ballast, sub-ballast and subgrade are considered as elastic materials in the analysis of stresses in the subgrade (Powrie et al., 2007). The first two layers comprise of ballast and sub-ballast to protect the subgrade not only from excessive deformation caused by train loading, but also progressive deformation (Powrie et al., 2007; Powrie et al., 2008). The ballast resists vertical, lateral and longitudinal forces applied to maintain the track in its correct position, to enhances energy absorption, resiliency and drainage, to reduce applied stresses and to assist maintenance work (Burrow et al., 2007; Lackenby et al., 2007). Sub-ballast (normally a granular material between ballast and subgrade) helps drainage and reduces track-related stresses to an acceptable level. The sub-ballast also stops upward migration of subgrade materials and acts as a barrier to interpenetration of subgrade and ballast additionally it protects subgrade abrasion caused by the ballast (Burrow et al., 2007).

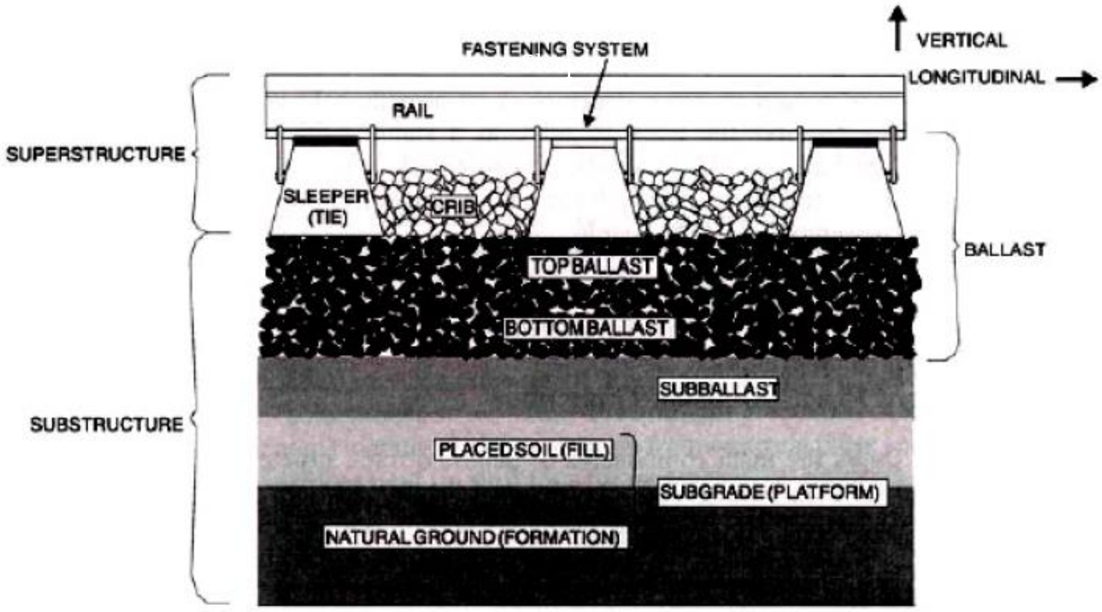


Figure 2.1 Conventional railway track components (Selig and Waters, 1994)

Li and Selig (1998) noted that subgrade performance depends on both the maximum axle load and repeated loading. The most common failure of subgrade is excessive plastic deformation and progressive shear failure due to repeated loads. The main aim in designing railway structure is to protect subgrade during the track's design life by providing an adequate thickness of the ballast and sub-ballast, if this protection is not achieved excessive deformation can cause significant maintenance costs (Burrow et al., 2007; Li and Selig, 1998).

2.3 Subgrade problems

Subgrade performance depends on several factors, particularly: cyclic loading, stress/ strain distribution, subgrade properties and environmental factors. Sadeghi and Askarinejad (2007) reported that the allowable annual tonnage of a high-quality subgrade track is four times higher than that of one with a poor-quality subgrade. Brough et al (2003) suggested that track performance and subsequent maintenance are reliant on the magnitude and variation of subgrade stiffness. Gräbe and Shaw (2010) identified three main reasons for the substructure failure in their investigation: (i) stabilised sub-ballast cracking as a result of inadequate support from the subgrade, (ii) weathering and decomposition of the substructure as a result of moisture content variations, temperature and cyclic loading and (iii) improper drainage and consequent saturation which decreased stiffness and strength.

The major railway subgrade problems considered are repeated train loading and fine-grained soil and excessive moisture content (Li and Selig, 1995). Li and Selig (1995) categorised the three major causes of subgrade problems as:

(i) Load factor: generally, two types of loads which are material self-weight and repeated dynamic loading. The first type of loading can be responsible for consolidation settlement or massive shear failure (deep rotational failure) if the track's embankment is not designed properly. On the other hand, the repeated loading has two characterise features, the magnitude of the individual dynamic wheel load and number of repetitions. This is the major concern for the subgrade.

(ii) Soil factor: this can cause major subgrade problems, depending on soil characteristics. A subgrade problem is not generally associated with coarse-grained soil (sand and gravel) but fine-grained soil (clay and silt), because of latter's lower strength and permeability. The influence of soil type on the subgrade performance depends on moisture content. Some soils can drain water very quickly and some cannot. Therefore, subgrade problems are commonly associated with fine-grained soil, which is susceptible to reduce shear strength and stiffness as a result of increasing moisture content.

(iii) Environmental factors: soil properties and behaviour change with moisture content and temperature. Subgrade can be wet or saturated by the infiltration of water from the surface or from groundwater. According to past research, the presence of groundwater from within 6m influences the subgrade moisture content; it is a major factor which is responsible for moisture content formation and subsequent problems.

Soil temperature concerns freezing and thawing. The freezing of soils, resulting from the combination of temperature, soil suction, soil permeability and availability of water and ice lenses, causes ground heave. On the other hand, thawing weakens the soil due to the presence of excess water from ice lenses.

The four major types of subgrade failures are discussed below. In addition to the four major types of subgrade problems, other subgrade problems may also lead to subgrade failure. Table 2.1 summarises subgrade problems, their causes and features.

2.3.1 Massive shear failure

Massive shear failure occurs when the substructure fails catastrophically under driving forces (the weights from the train, the superstructure and the unbalanced portion of the substructure) (Selig and Waters, 1994). The loading that causes massive shear failure is the substructure self-weight, track superstructure and train weights. Figure 2.2 shows a massive shear failure. A shear surface or slip surface refers to the strength characteristics of the subgrade that control the resistive strength of the substructure to massive shear failure. Massive shear failure is likely to be a problem when the subgrade strength is reduced due to an increase of water content resulting from heavy rainfall or flooding.

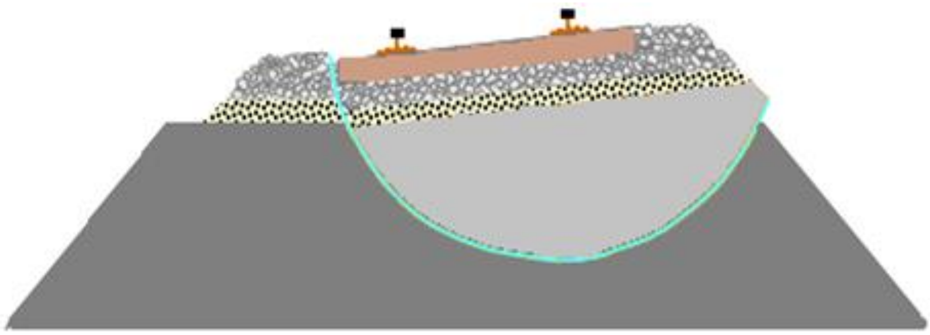


Figure 2.2 Massive shear failure of the track (after Selig and Waters, 1994)

2.3.2 Progressive shear failure

Progressive shear failure (also known as cess heave or general subgrade failure) is the plastic flow of the soil which occurs from the overstressing caused by repeated loading, where the subgrade deforms a small amount horizontally with applied vertical loading, the subgrade materials being squeezed sideways and upwards (Selig and Waters, 1994; Li and Selig, 1995) as seen in Figure 2.3. Generally, as the moisture content of the subgrade increases, the shear strength of soil reduces and the pore pressure increases with repeated loading. This type of problem is often associated with fine-grained soil with high clay content. The addition of more ballast can reduce subgrade stress.

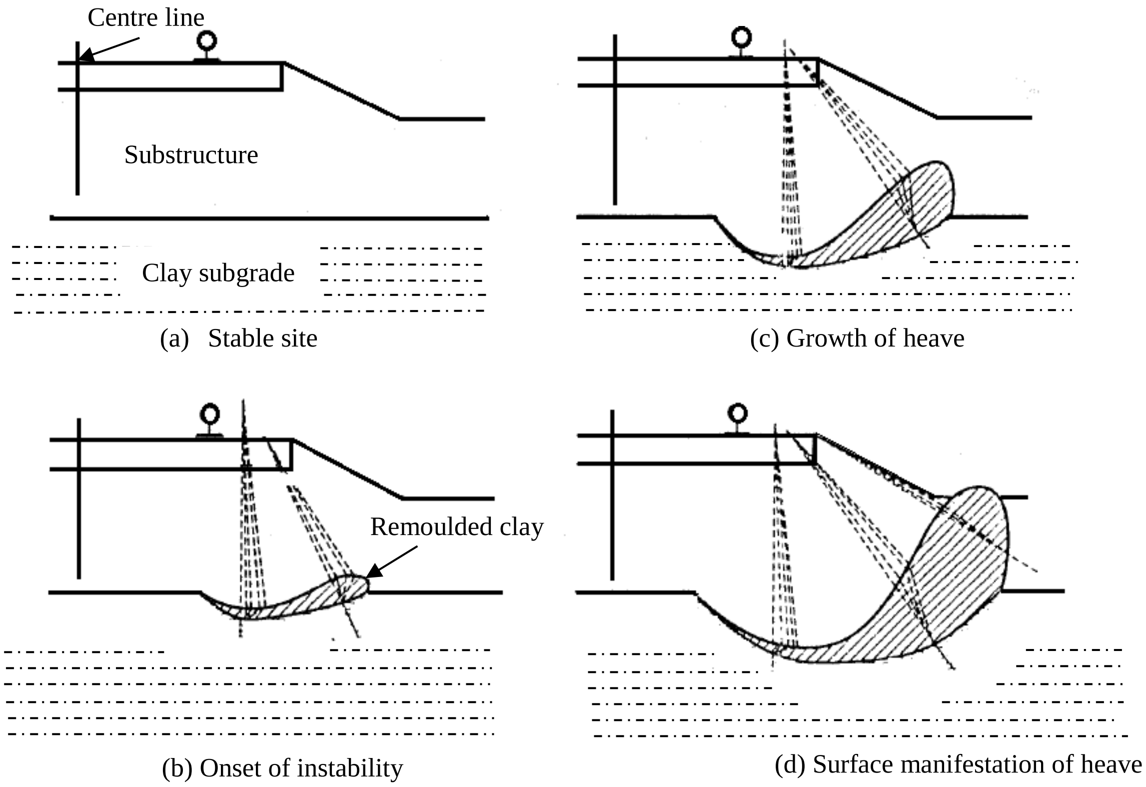
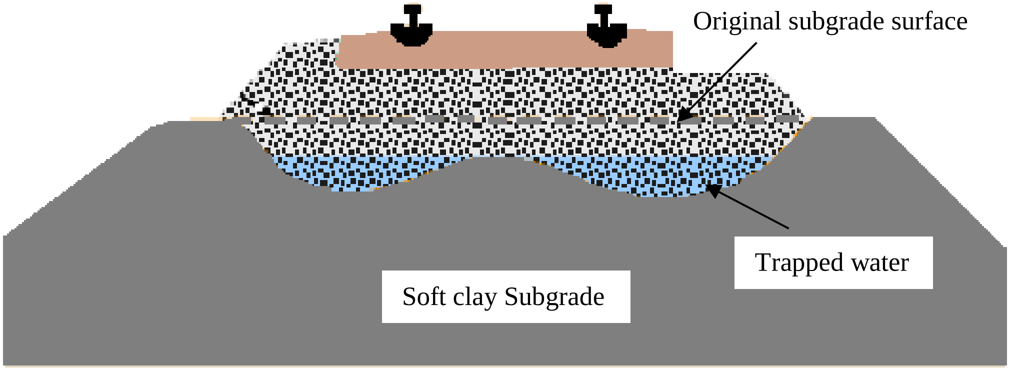


Figure 2.3 Progressive shear failure (after Selig and Waters, 1994)

2.3.3 Excessive plastic deformation

Another problem associated with soft clay subgrade is excessive plastic deformation. This problem not only includes the vertical component of progressive shear deformation but also the vertical deformation which occurs due to the progressive compaction and consolidation of subgrade under cyclic load (Li and Selig, 1995). Figure 2.4a presents a diagram of the excessive plastic deformation and Figure 2.4b shows an example of a ballast pocket across the track. The loss of track elevation due to excessive plastic deformation in the subgrade requires to be added by ballast, which increases the overall ballast depth. A ballast pocket can develop under the rail, as a result of trapped water, due to the continuous replacement of subgrade by ballast. Furthermore, ballast can be contaminated with the subgrade soil; hence, ballast characteristics can be degraded.



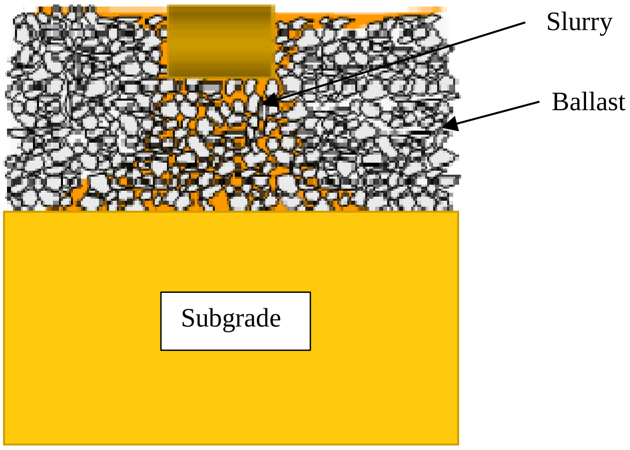
(a)

(b)

Figure 2.4 (a) Schematic subgrade plastic deformation (after Li and Selig, 1995) & (b) Water flowing from a ballast pocket (ARTC, 2003)

2.3.4 Subgrade attrition with mud pumping

Subgrade attrition by ballast, in the presence of water, is caused in the formation of slurry in the ballast-subgrade interface. Figure 2.5a presents a schematic diagram of subgrade attrition with mud pumping. This type of phenomena occurs when ballast is placed directly on fine-grained soil or soft rock. The formation of slurry at the ballast-subgrade interface, which is then pumped upwards into the ballast is shown in 2.5b. The overloading of the traffic at the ballast-subgrade interface causes the wearing-away of the soil or rock subgrade surface. The presence of water and the worn materials form mud, which pumps upwards into the ballast by the repeated train loading. This process induces track settlement and reduces the drainage capacity of the ballast.



(a)

(b)

Figure 2.5 (a) Subgrade attrition diagram (after Selig and Water, 1994) & (b) Overloading subgrade and ballast penetration (Wenty, 2005)

Table 2.1 Subgrade problems and their characteristics (after Li and Selig, 1995)

Type		Causes	Characteristics
Repeated Traffic Loading	Progressive shear failure	• Repeated over stressing • Fined grained soils • High water content	• Squeezing near subgrade surface • Heaves in crib and shoulder • Depression under ties
	Excessive plastic deformation	• Repeated loading • Soft or loose soil	• Differential subgrade settlement • Ballast pockets
	Subgrade attrition with mud pumping	• Repeated loading of subgrade by ballast • Contact between ballast and subgrade • Clay rich rocks or soils Water presence	• Muddy ballast • Inadequate sub-ballast
	Liquefaction	• Repeated loading • Saturated silt and fine sand	• Large displacement • More severe with vibration • Can happen in sub ballast
The weight of the train , track and subgrade	Massive shear failure (slope stability)	• Weight of train, track and subgrade • Inadequate soil strength	• High embankment and cut slope • Often triggered by increase in water content
	Consolidation settlement	• Embankment weight • Saturated fine-grained soils	• Increased static soil • Stress as from newly constructed embankment
Environmental factors	Frost action (heave and softening)	• Periodic freezing temperature • Free water • Frost susceptible soils	• Occur in winters/spring period • Rough track surface
	Swelling/Shrinkage	• Highly plastic soils • Changing moisture content	• Rough track surface
	Slope erosion	• Running surface and subsurface water • Wind	• Soil washed or blown away
	Soil collapse	• Water inundation of loose soil deposits	• Ground settlement

2.4 Track settlement

Track settlement is a function of the number of loading cycles and the magnitude of the loading. Track settlement occurs as a result of permanent deformation in the ballast, sub-ballast and subgrade soil, due to cyclic loading; it also depends on the quality and behaviour of ballast and subgrade (Selig and Waters, 1994; Dahlberg, 2001; Dahlberg, 2004). In a newly constructed track, significant contributions to settlement may come from sub-ballast and subgrade as they have not previously been experienced to considerable traffic load. On the other hand, for track that has been in service for a long time, the layers that are not distributed by tamping will usually contribute only a minor part to further settlement. The upper ballast which is distributed periodically by tamping is generally the primary source of track settlement (see Figure 2.6) (considered a high-quality subgrade).

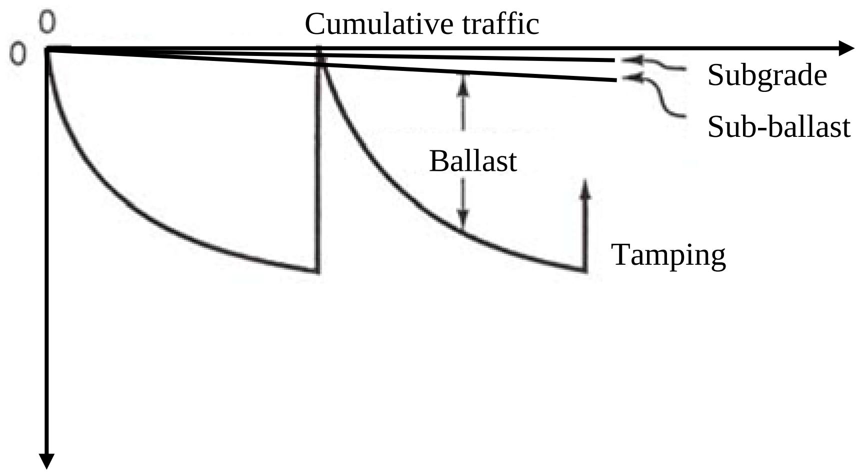


Figure 2.6 Substructure contributions to settlement (Brown and Selig, 1991)

Dahlberg (2010) reported that the rate of degradation and settlement depends on the severity of the stiffness variation; deterioration of the track's geometry causes an increase of the track interaction forces which speeds up the track degradation rate. Selig and Li (1994) stated that the major cause of track deterioration is settlement of the substructure. A track with a stiff subgrade support has a higher track stiffness than a track with a soft subgrade (see Figure 2.7). The track settlement behaviour for the different subgrade modulus is shown in Figure 2.8.

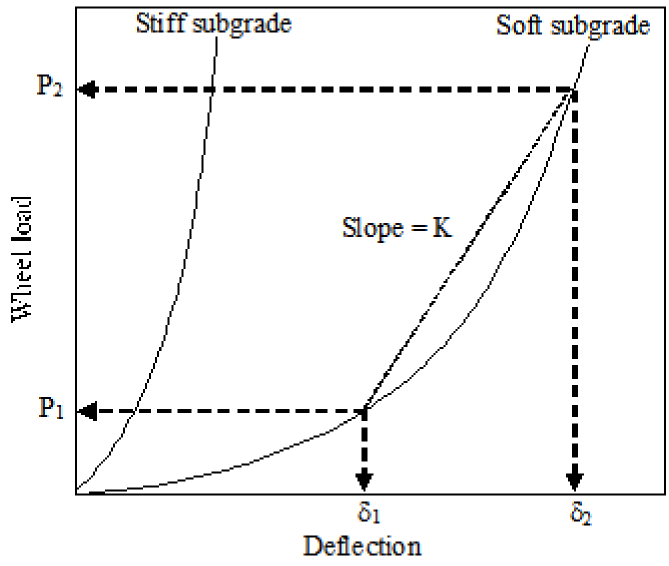


Figure 2.7 Comparison of track modulus between stiff and soft subgrades (Li and Selig, 1995)

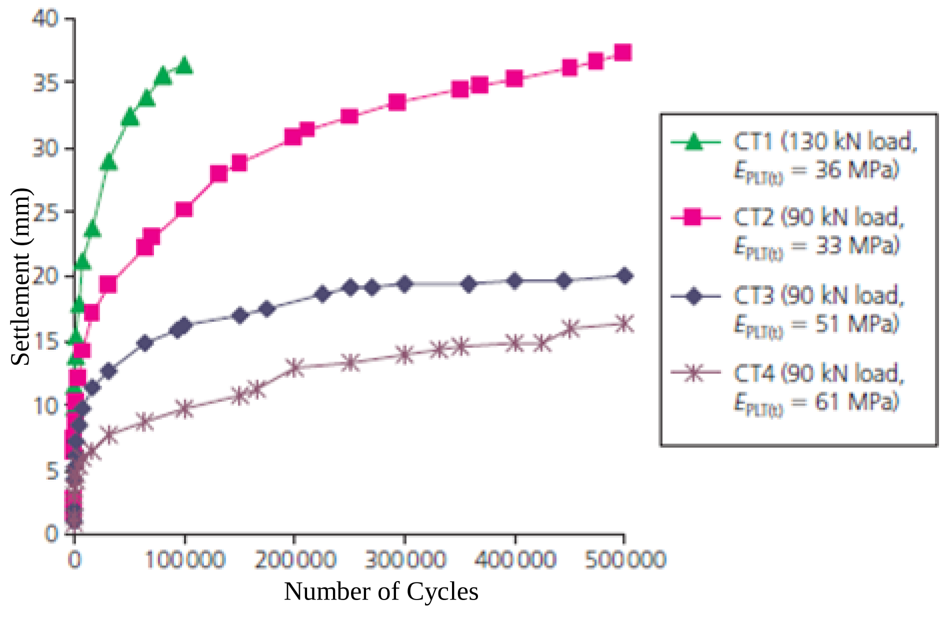
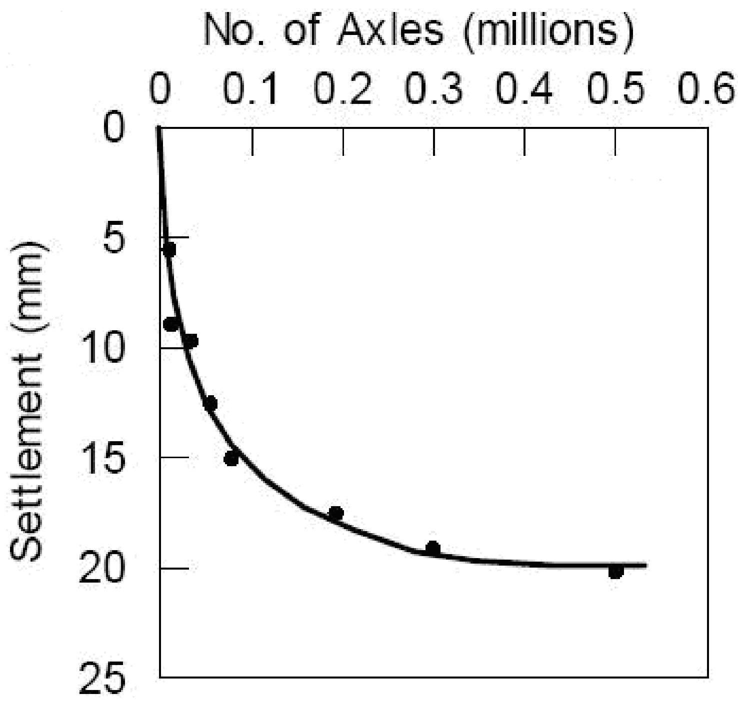
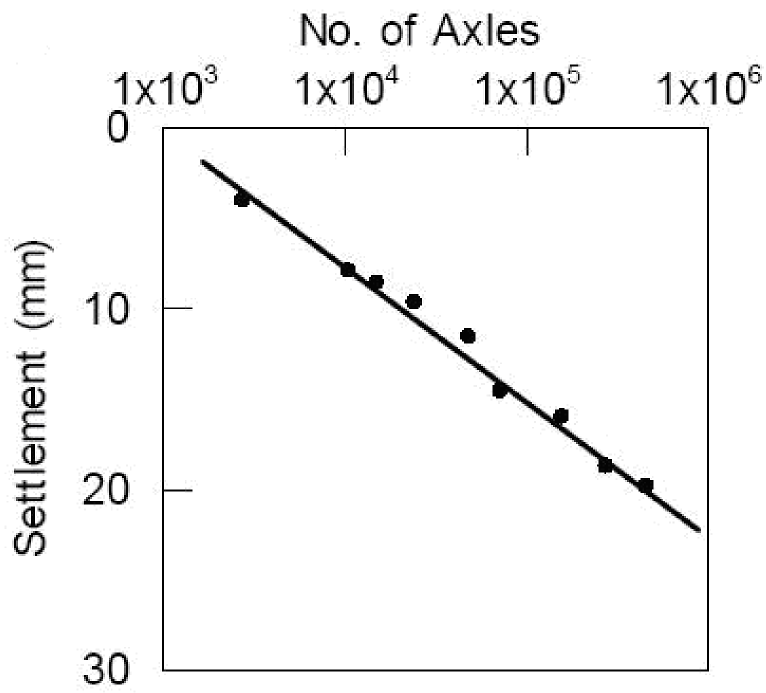


Figure 2.8 Track settlement for various subgrade stiffness (Kennedy et al., 2012)

Permanent deformation in ballast under cyclic loading is accumulated over an increasing number of load cycles. Shenton (1974) documented that track settlement immediately after tamping increased at a decreasing rate relative to the number of axles is shown in Figure 2.9a. Furthermore, he reported that the track settlement would be approximated by a linear relationship with the logarithm of loading cycles (Figure 2.9b). It was observed by other researchers (Raymond and Williams, 1978; Brown and Selig, 1991; Selig and Waters, 1994) that the track settlement or vertical strain accumulated approximately logarithmically with the number of loading cycles.



(a)



(b)

Figure 2.9 Settlement of track after tamping (a) on plain scale, (b) on semi-logarithmic scale (Shenton, 1974)

Several researchers reported that the permanent settlement of ballast can comprise of two stages settlement (Jeffs and Marich, 1987; Indraratna et al., 1998; Indraratna and Ionescu, 1999; Dahlberg, 2001). The first stage settlement is considered the initial ballast densification to a higher density and rapid settlement occurrs. In the second stage, the settlement is slower and approximately linear. Dahlberg (2001) pointed out two major phases of ballasted track settlement which are:

1. Stage-1: track settlement is comparatively fast directly after tamping until the ballast densifies.
2. Stage-2: This phase is relatively slow and displays a linear relationship between settlement and time.

The second phase of settlement occurs due to fundamental mechanisms of ballast and subgrade performance; (i)-(iv) below relate to ballast and subgrade densification related and (v)-(vii) refer to the inelastic behaviour of ballast and subgrade.

i. After phase-1 above, densification continued by rearrangement of particles due to cyclic loading.
ii. Ballast moving down into the sub-ballast or subgrade soil. The sub-ballast/subgrade also migrates into the ballast voids.
iii. The ballast can fracture as a result of repeated loading or environmental factors.
iv. Abrasive wear can influence volume reduction as ballast particles can reduce in volume due to abrasive wear.
v. Deformations may not recover upon unloading due to micro-slip between ballast particles at loading.
vi. The ballast and subgrade movement under the sleepers induces track settlement.
vii. Both lateral and longitudinal movement of the sleepers, which push the ballast away from beneath the sleepers, causes the sleepers to move down into the ballast.

2.4.1 Track settlement models

Various models have been developed for predicting track settlement under cyclic loading. A review of these models can be found in Kennedy (2010), where GRAFT data were compared with other models.

Hettler (1984) suggested that the accumulation of permanent deformation was proportional to the logarithm of the number of cycles of load and expressed as:

$$S_N = S_1(1 + C'logN) \tag{2.1}$$

where the factor c' takes values between 0.25 and 0.55.

Sato (1995) suggested that the track settlement (y) under cyclic loading (x) can be expressed as:

$$y = \gamma(1 - e^{-\alpha x}) + \beta x \tag{2.2}$$

where x is the loading of the track (Figure 2.10). The x can be expressed either as a number of load cycles on the track or as tonnage carried by the track. The α, β and γ are constants parameters describing the short-term and long-term settlement behaviour. The first part of equation (2.2), i.e.$\gamma(1 - e^{-\alpha x})$, describes the short-term settlement of the track immediately after a tamping; the factor γ gives the severity (size) of the settlement and the factor α indicates how quickly the initial part of the settlement attenuates.

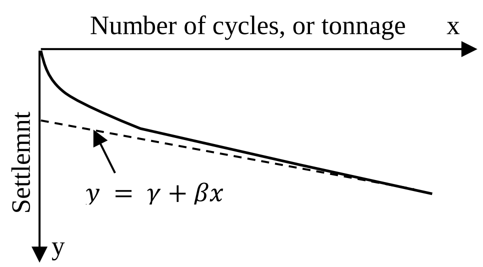


Figure 2.10 Track settlement as a function of loading x

Shenton (1985) proposed a settlement model based on laboratory and field experiments which is:

$$y = K_1 N^{0.2} + K_2 N \tag{2.3}$$

where K_1 and K_2 are material constants and the linear term (second part of the equation) only becomes significant for values of N above 10^6. Their numerical values depend on a number of factors, such as axle load, sleeper spacing, rail section and track and foundation stiffness.

Li and Selig (1996) reported that the most commonly used power model for predicting cumulative plastic strain in soil under cyclic loading is:

$$\varepsilon_p = AN^b \tag{2.4}$$

where ε_p is the cumulative plastic strain (%), *N* is the number of cyclic loading and *A* and *b* are the two parameters depending on soil type, soil properties and stress state.

Li and Selig (1996) proposed a relationship for *A*, where a soil strength parameter under static loading has been used to represent indirectly the influence of the soil's physical state, which is:

$$A = a\left(\frac{\sigma_d}{\sigma_s}\right)^m \tag{2.5}$$

where *a* and *m* are material parameters, σ_d is the deviator stress and σ_s is the soil static strength (compressive strength under monotonic loading).

Selig and Waters (1994) proposed a model based on field measurement and box tests which distinguished between the contributions of the ballast and subgrade settlement. The model is:

$$y = k_1\, N^b + k_2\, N^c \tag{2.6}$$

where the first part of the equation describes ballast settlement and the second part subgrade settlement; *y* is the track settlement in inches; K_1 is the ballast settlement from the first cycle; *b* is a ballast exponent; K_2 is the subgrade settlement after the first cycle and *c* is a subgrade exponent. From the field study, K_1 = 0.027 to 0.042, *b* = 0.21 to 0.22, K_2 = 0.0014 to 0.00052 and *c* = 0.37 to 0.52.

Shi (2009) developed a settlement equation based on the ballast cyclic triaxial tests which was validated by railway test facility (RTF)- a full-scale testing facility and composite element test (CET). The proposed model is:

$$\varepsilon_a = k \cdot N^c \qquad (2.7)$$

where ε_a is the permanent axial strain, k is the first cycle, N is the number of load cycles and c is introduced as a peak stress ratio which can be expressed as:

$$c = 10^{-9} \times (2\times10^7)^\wedge((q/p')_p)+0.05 \qquad (2.8)$$

k can be estimated by:

$$k = s.\ F^{(q/p')}{}_{max} + \mathrm{b} \qquad (2.9)$$

where

s	is a scaling factor and recommended as: $s = 5\times10^{-6}$
F	is a coefficient related to confining pressure $F = 5\times\sigma_3' + 400$
$(q/p')_{max}$	is the maximum stress ratio
b	*is a coefficient related to confining pressure and defined as:*
	$b = 0.008\times1.08^{\sigma_3'}+0.48$ (Unit of σ_3': kPa)

Kennedy et al. (2012) proposed a model for predicting track settlement for the GRAFT which is:

$$y = K_1 N^{0.23} \qquad (2.10)$$

where y is the settlement (mm) after N cycles and K_1 is a constant depending on applied load and subgrade modulus.

Figure 2.11 presents a comparison among the track settlement models and the GRAFT subgrade modulus model. The GRAFT subgrade modulus model predicts settlements that are almost identical to both the Selig and Waters (1994) and Shenton (1985) equations.

However, this particular GRAFT subgrade modulus model does not distinguish between ballast and subgrade and hence the Shenton (1985) model is more appropriate for direct comparison as it also does not distinguish between these components.

For the tangent modulus, the equation can be written as:

$$y = 1281t^{-1.3342}N^{0.23} \quad (2.11)$$

where K_1 = $1281t^{-1.3342}$ and $t = \frac{E_{PLT}}{P} = \frac{stiffness}{pressure}$

where t is the track parameter and p is the sleeper-ballast contact pressure on the middle sleeper in GRAFT

For the reloading modulus, the equation can be written as:

$$y = 1395t^{-1.2707}N^{0.23} \quad (2.12)$$

where K_1 = $1395t^{-1.2707}$and $t = \frac{E_{PLT}}{P}$

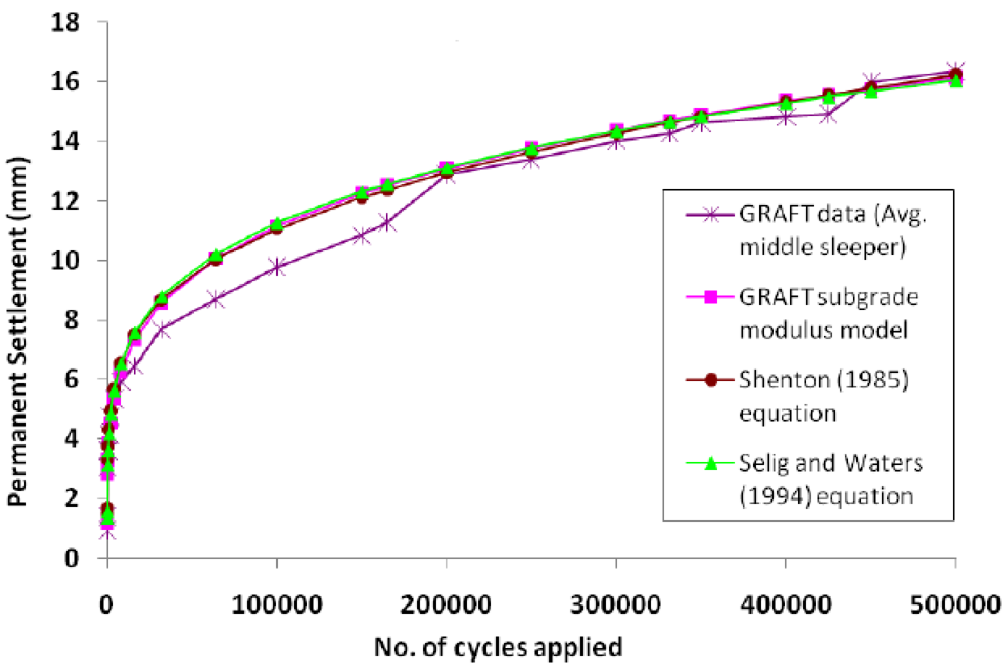


Figure 2.11 Comparison of settlement models with GRAFT subgrade modulus model (Kennedy, 2010)

2.5 Track damages due to uncontrolled overflow

In 1889, H. Frazier of the Chesapeake and Ohio Railway wrote: "*The stability of track depends upon the strength and permanence of the roadbed and structures upon which it rests: whatever will protect them from damage or prevent premature decay should be carefully observed"*. The worst enemy is "water", and the further it can be kept away from the track or the sooner it can be diverted from it, the better the track will be protected (Wenty, 2005). Flooding causes significant problems in the railway infrastructure and is the biggest challenge for the engineers whose job is to keep the railway track functional.

With reference to Figure 2.12, water can enter the track substructure in three ways (i) precipitation (rain and snow), (ii) water flowing down adjacent slopes and (iii) through underlying layers, as water can seep upwards from the subsurface (Selig and Water, 1994). A saturated state of subgrade increases maintenance costs significantly due to (Selig and Water, 1994): (i) increasing of pore pressure under cyclic load which causes an increase in plastic strain accumulation, (ii) loss of strength, (iii) subgrade attrition and slurry formation (iv) volume change, (v) mud pumping (vi) frost heave/thaw softening and (vii) ballast degradation and sleeper attrition from slurry abrasion. Selig and Cantrell (2001) indicated that the track is yet considered to be operational if the water is at the base of the surface level of the track. However, if the water reaches the base of the sleeper then the operations are likely to halt. Generally, flooding significantly changes the behaviour of subgrade materials behaviour; the subgrade becomes soft with a loss of shear strength and bearing capacity (Clarke and Cosby, 2007).

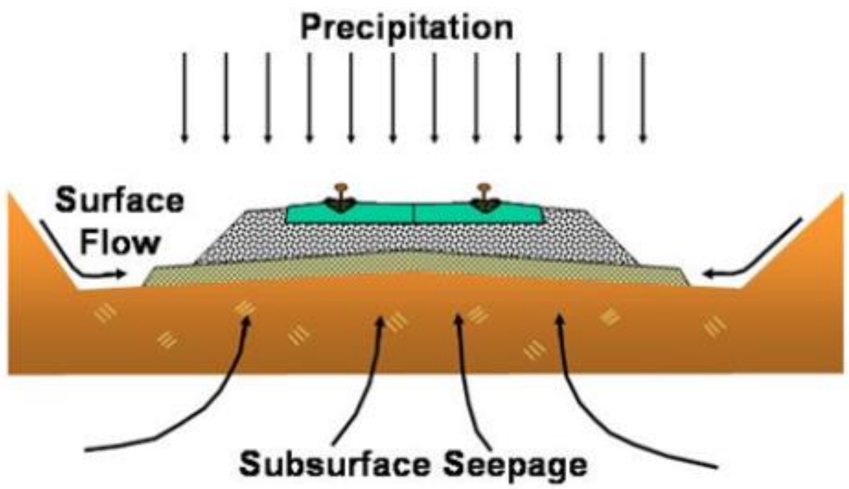


Figure 2.12 The three sources of water entering in track substructure (after Selig and Waters, 1994)

2.5.1 Causes of flooding

Flooding is normally caused by natural weather events such as heavy rainfall over a short period, prolonged, extensive rainfall and high-tide combined with stormy conditions. Network Rail (2011) documented some common types of flooding which impact on rail track performance:

Tidal flooding

Storms with high wind speeds cause high and powerful waves. Low-pressure fronts cause sea-levels to rise above normal levels; high tide levels vary through the lunar and solar cycles and, when superimposed upon other tidal variations, exceptionally high tides result.

Fluvial flooding

Fluvial flooding occurs where a river bursts or overtops its banks and floods the areas around it. This type of flooding is generally caused by prolonged, extensive rain and can be exacerbated by melting snow from within catchment areas further upstream.

Flash flooding

A flash flood describes a very fast moving and unexpected event which occurs due to heavy rain. It also arises if flood defences fail or drainage systems are insufficient. Flash flooding very difficult to predict and it is caused by localised weather conditions.

Groundwater flooding

When the water levels underneath the ground rise above normal levels approaching the surface, ground water flooding is a likely result, due to prolonged period of rainfall. Such flooding can last for weeks to months.

Flooding from sewers

The sewers flooding may happen due to system failure. This flooding is a combination of storm and foul sewers and when, their capacity is exceeded due to a large amount of water run-off in a short time.

Flooding from manmade infrastructure

Failure of man-made structures such as canals and reservoirs failure can cause flooding. Industrial activities, water mains and pumping station failure also can cause flooding.

Tables 2.2 and 2.3 present the possible structural damage directly and indirectly.

Table 2.2 Description of infrastructure damage to railway lines/cross section and possible indirect impacts (Moran et al., 2010)

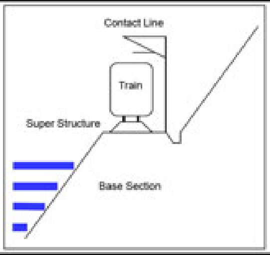
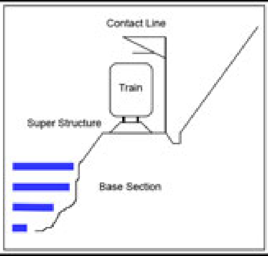
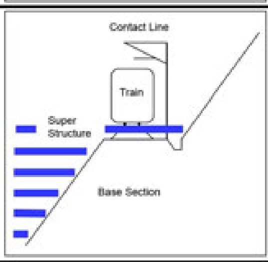
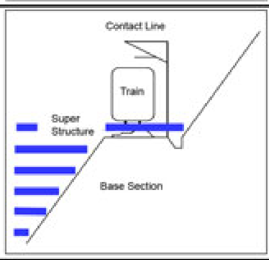
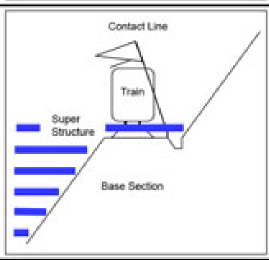
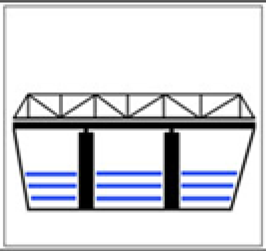
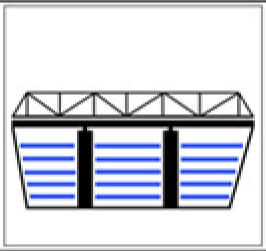
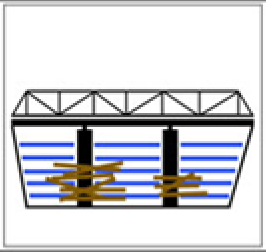
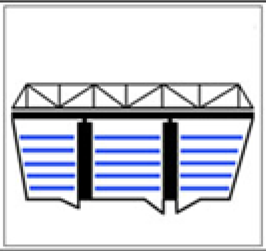
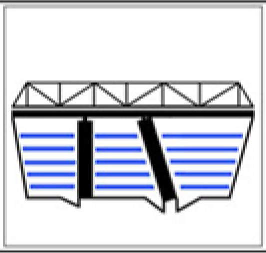
#	Damage grade	Description of direct structural damage	Description of possible indirect effects
1		Flooding reaches the base section without any notable damage.	none
2		Flooding reaches the base section. Erosion occurs.	Possible operational limitations (slower speeds, delays).
3		Track super structure is flooded.	Track is closed. Detours or replacement transport services are necessary.
4		Erosion of the track super structure. Complete reconstruction is necessary.	Detours or replacement transport services for several days are necessary.
5		Additionally the overhead contact line, signals etc. are damaged.	Detours or replacement transport services for several weeks are necessary.

Table 2.3 Structural damage to railway bridges and possible indirect impacts (Moran et al., 2010)

#	Damage grade	Description of direct structural damage	Description of possible indirect effects
1		No adverse effects due to flooding.	none
2		No restrictions, maintenance necessary in the course of the next inspection.	none
3		Log jam, removal is required.	In general no operational limitations. Speed reductions or short closures are possible during the removal of the log jam.
4		Formation of a scour. Repair is necessary.	Rail operations are restricted. Detours or replacement transport services may be necessary.
5		Scour or major structural damage to the object. Bearing strength his no longer guaranteed. Bridge is completely closed.	All rail operations are closed for many days or weeks.

2.5.2 Track drainage

'*Water is the greatest enemy to the civil engineers*' was said by one of the greatest civil engineers Thomas Telford. Ghataora and Rushton (2012) reported that it is almost impossible to stop water entering in a ballasted track. Therefore, it is important to drain away water from the track as soon and as efficiently as possible, in order to ensure that the track can be safely supported by its subgrade. An effective track drainage system drains water from the track quickly, limiting the water from accessing the subgrade as well as directing surface flow from the track (Heyns, 2000; Indraratna et al., 2013a). Track drainage is an important factor regarding track performance and maintenance (Heyns, 2000; Selig and Cantrell, 2001; Bonnet, 2005; Ghataora and Rushton, 2012). The main purpose of the drainage is to divert the water from the track to keep the track safe.

Selig and Cantrell (2001) pointed out that, to achieve appropriate drainage, does not involve simply digging a cross trench and drainng the water from the track. If not used properly, inefficient drainage can allow water into the subgrade, threby causing more softening. Track drainage is a complicated problem as so many factors are needed to be considered, these include ballast condition, subgrade surface slope, gradation of sub ballast, ditch or pipe longitudinal slope, expected rainfall effect and clay behaviour (Heyns, 2000).

Ghataora and Rushton (2012) reported that water can be retained in the nearest the track drain for more than a week; consequently, the water can affect on track performance and therefore more maintenance will be required. Track drainage is designed to maintain water flow and levels around the track; therefore, an appropriate track drainage system is required, the designers of which need to have a comprehensive knowledge of the material behaviour. Ghataora and Rushton (2012) also reported that the drainage pipes in the cess area are designed in Britain to run full bore for a two year rainfall event, but the problem started with a five years event. Figure 2.13 shows rail tracks experiencing problem from inadequate drainage.

Figure 2.13 Poor track drainage creates ponding near the track (Lackenby, 2006)

In section 2.5 above, it was stipulated that the water can enter a track from three sources (i) precipitation, (ii) surface flow from nearby areas and (iii) groundwater flow (Selig and Water, 1994). Therefore, to drain water from the track a complete drainage system is required to control the water from all three sources (Heyns, 2000). Track drainage entails of two types: (i) surface drainage and (ii) subsurface drainage.

2.5.2.1 Surface drainage

Surface drainage removes surface runoff before the water enters the track and directs the water out and away from the track. There are three main types of surface drainage which are described below:

Cess drains:

Cess drains are located at subgrade formation level at the side of the tracks. This drain's main function is to protect the subgrade formation and to keep dry. Cess drains are generally found in cuttings where water running off the formation cannot freely drain away (Tzanakakis, 2013). Figure 2.14 shows a schematic diagram of cess drains.

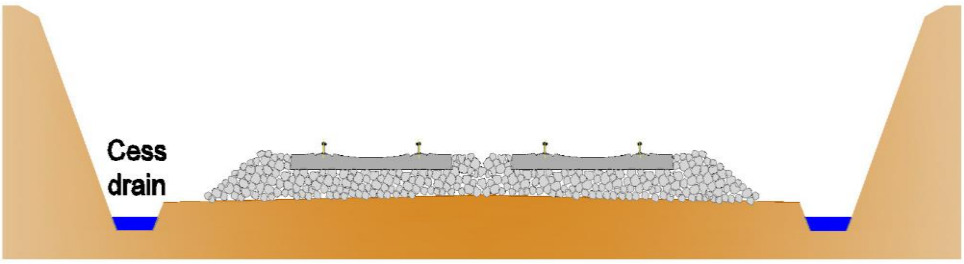


Figure 2.14 Typical location of Cess drains (Tzanakakis, 2013)

Catch drains:

Catch drains (also known as top drains) are generally located on the uphill of a cutting to catch water flowing down the hill and to remove it before it reached the cutting is shown in Figure 2.15. Catch drains may be used alongside tracks that cut across a slight downhill grade (Tzanakakis, 2013). The main function of catch drains is to intercept overland flow or runoff before it reaches the track.

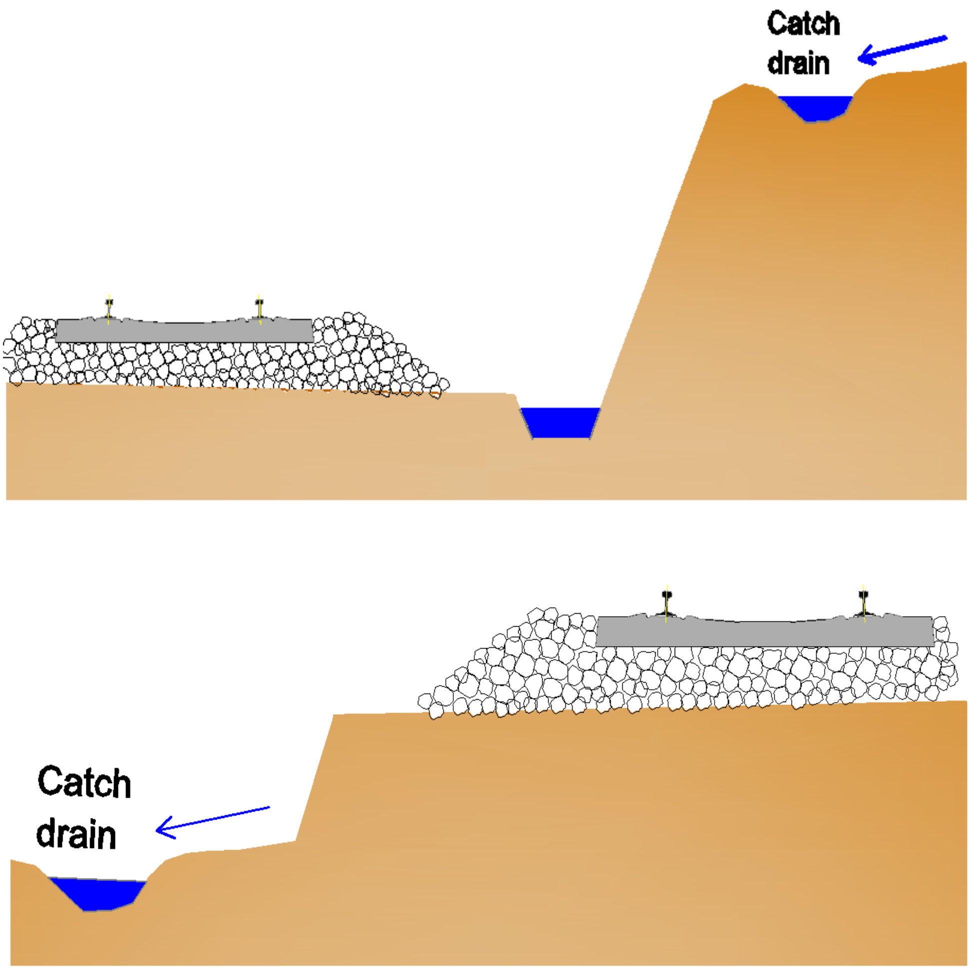


Figure 2.15 Typical catch drains (Tzanakakis, 2013)

Mitre drains:

Mitre drains (also known as spoon or offshoot drains) are connected to cess and catch drains to allow an escape water from these drains, as shown in Figure 2.16 (Tzanakakis, 2013). These drains should be provided at regular intervals to remove water before it slows down and deposit any sediment.

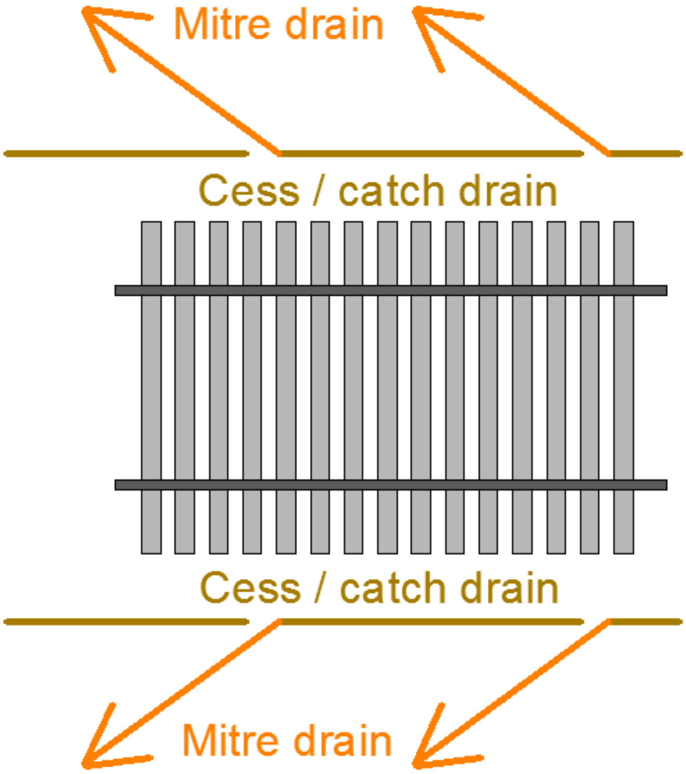


Figure 2.16 Mitre drains (Tzanakakis, 2013)

2.5.2.2 Subsurface drainage

Subsurface drainage systems are designed to deal with surface runoff, ground water and seepage, as well as water collected from other drainage systems. Subsurface drainage should be provided in locations where the water table is at or near earthworks level (Tzanakakis, 2013). Subsurface drains are provided where surface drainage cannot be provided due to some restriction or lack of available fall due to outlet restrictions; such as, platforms, cuttings, junctions and bridges. Subsurface drainage systems are more complicated than the surface drainage systems. It requires a detailed hydrological and geotechnical investigation. Subsurface drainage systems are effective for draining water from soil, draining water, from ballast pockets, draining from cracks in the ground, lowering the ground water elevation and intercepting water flowing toward the track. Figures 2.17-19 present the subsurface drainage systems.

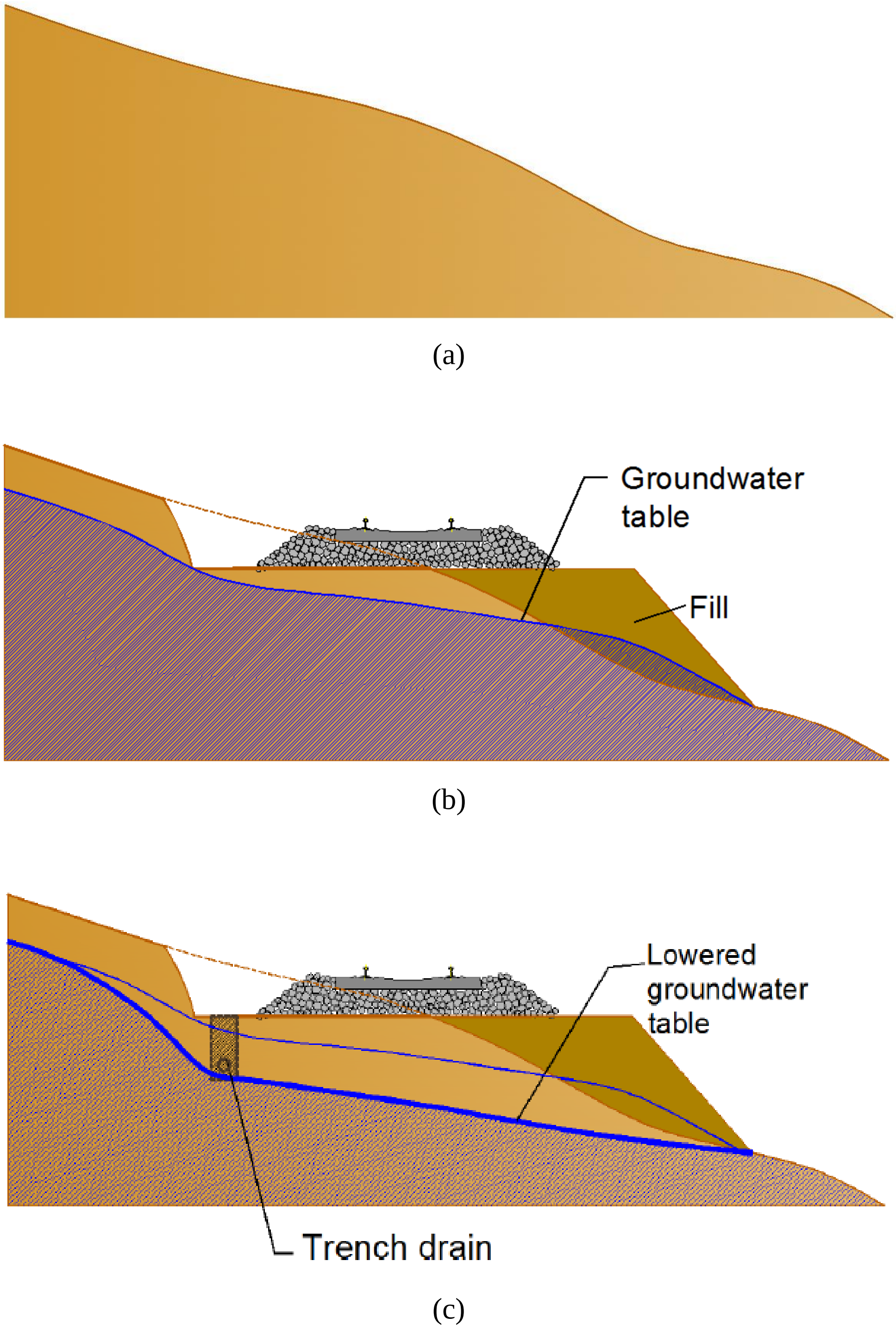


Figure 2.17 (a) a slope before of a track construction, (b) high ground water table before installing trench drain and (c) lowered ground water after trench drain installation (Tzanakakis, 2013)

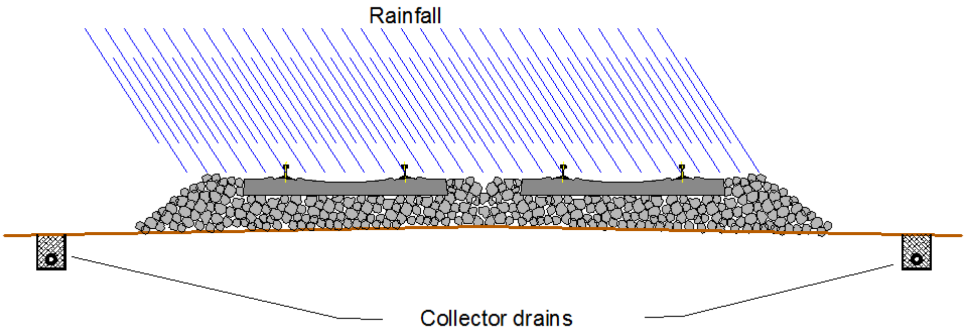
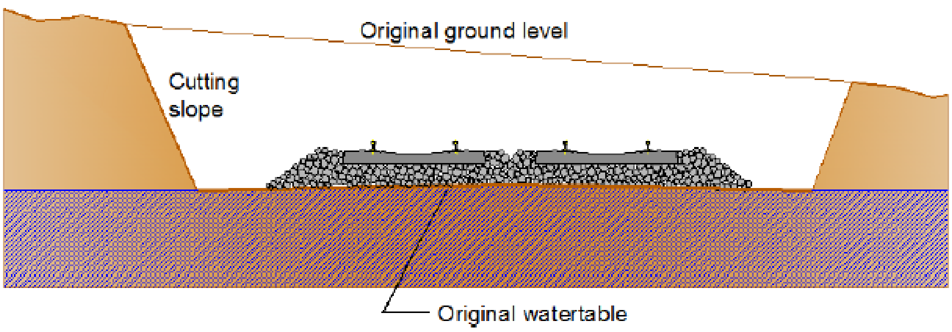
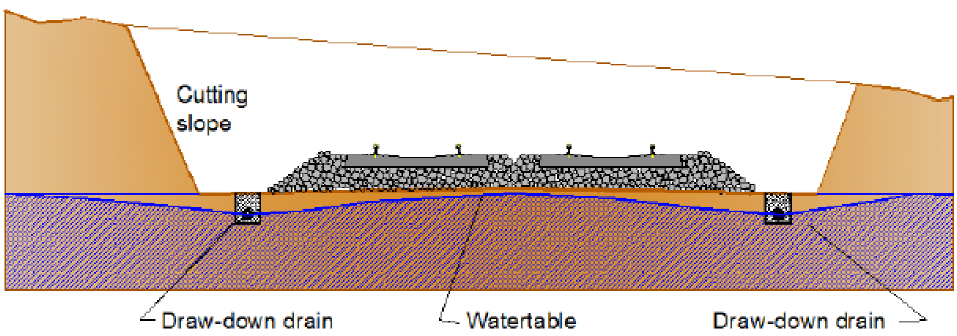


Figure 2.18 Collection of water seeping into the ballast structure (Tzanakakis, 2013)



(a)



(b)

Figure 2.19 (a) Before the drainage and (b) lowering water table after drainage (Tzanakakis, 2013)

2.5.3 Role of Geosynthetics and sand blanket

The Geosynthetic products in the UK have been used since the introduction of geotextiles in 1970 as separators between ballast and formation; a use that has increased steadily (Sharpe and Caddick, 2006). The aim of application of Geosynthetics is to improve the bearing capacity of subgrade soils, to prevent contamination of the ballast by subgrade fines and to disperse the high pore water pressures built up by repeated loading. Indraratna et al.(2006) subdivided the function of Geosynthetics in to six categories:

(i) Separation
(ii) Reinforcement
(iii) Filtration
(iv) Drainage
(v) Moisture barrier/waterproofing and
(vi) Protection

Selig and Waters (1994) summarised the observations made by several researchers undertaking laboratory tests to evaluate the performance of geotextiles under cyclic loading:

- When the soil under the geotextile was clay, the repeated load caused the clay to pump through the geotextile, regardless of the geotextile; however, the rate of pumping varied with geotextile characteristics.
- The pumped slurry was formed at the contact points between the aggregate and clay through the geotextile with larger aggregates resulting in increased pumping.
- A sand layer in place of a geotextile was effective in preventing clay migration into the ballast; however, when used the geotextiles acted as an effective separator between the sand layer and ballast.

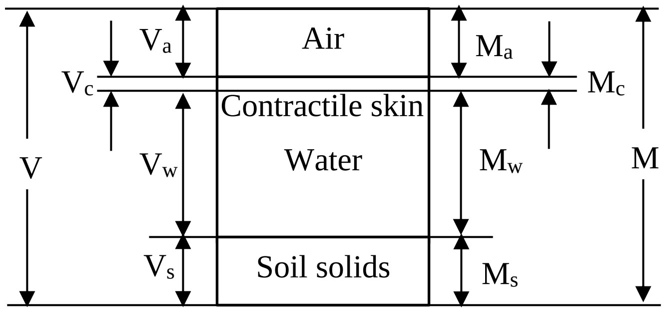
Indraratna et al. (2006) found (under laboratory cyclic testing to evaluate the performance of a geotextile compared to that of a conventional sand blanket) that the use of geotextiles reduced the migration of fine-grained material into the upper ballast layer, but they cannot prevent pumping of soils consisting of only clay-size particles. It was concluded that geotextiles were mostly suitable in the case of preventing pumping of fine soil that is broadly graded and contains significant amounts of sand-size particles. These findings were reiterated by Selig and Waters (1994), based on field observations in both the UK and US where combinations of geotextiles, geomembranes and geogrids all proved insufficient in preventing pumping without a sand blanket. Further evidence of geotextiles in use on railway track has shown that after they have been in the track for up to 48 months their permeability, tensile strength and transmisivity all decrease significantly (Selig and Waters, 1994).

It can be concluded from these findings that although a geotextile may reduce the rate of clay pumping it cannot prevent it alone and a sand blanket with a geotextile separator is a better combination. This is now the adopted system in the UK and within Network Rail standard (RT/CE/S/010, 1996), i.e. a non-woven geotextile separator is specified for use on track to separate a sand layer from ballast. A typical sand blanket construction requires a depth of 100mm and the grading specification for blanketing sand (Network Rail standard, RT/CE/S/033). However, geotextiles cannot replace the sand blankets themselves. Sharpe and Caddick (2006) stated there is further scope to investigate multifunctional geosynthetics (geocomposites) to try and reduce trackbed renewal costs and installation time incurred by laying sand blankets. They estimated that there are approximately 30 miles of sand blanket laid each year on UK railways. To fulfil the functions of a sand blanket a geosynthetic product must:

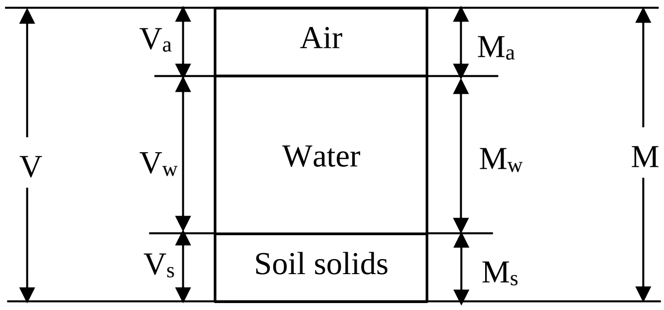
- Prevent upward migration of fine particles
- Resist abrasion under heavy dynamic loading
- Dry out any existing slurry
- Permit seepage of water from underlying subgrade
- Reduce stress on subgrade
- Prevent subgrade attrition by ballast
- Drain water from above and below

2.6 Unsaturated soil behaviour

Unsaturated soil is a three-phase system consisting of solids, gas (usually air) and liquid (usually water) phases (Bishop, 1959; Bishop and Blight, 1963; Lambe and Whitman, 1969; Fredlund and Rahardjo, 1993). In addition, a fourth phase should be considered when the soil is unsaturated namely, the air-water interface or contractile skin (Fredlund and Morgenstern, 1977; Fredlund and Rahardjo, 1993; Fredlund, 2006). Figure 2.20 shows a schematic diagram of unsaturated soil for the mass and volume for each phase.



(a)



(b)

Figure 2.20 Schematic phase diagrams for an unsaturated soil (a) rigorous four phase unsaturated soil system and (b) simplified three phase diagram (Fredlund and Rahardjo, 1993).

The pore pressure of unsaturated soil is negative. A saturated soil can turn into unsaturated if the pore pressure is reduced and air enters into the pores; just as unsaturated soil can be saturated when the water table increases (Fredlund and Rahardjo, 1993; Atkinson, 2007). Figure 2.21 presents the categorisation of soil mechanics and the subdivision of saturated and unsaturated soil.

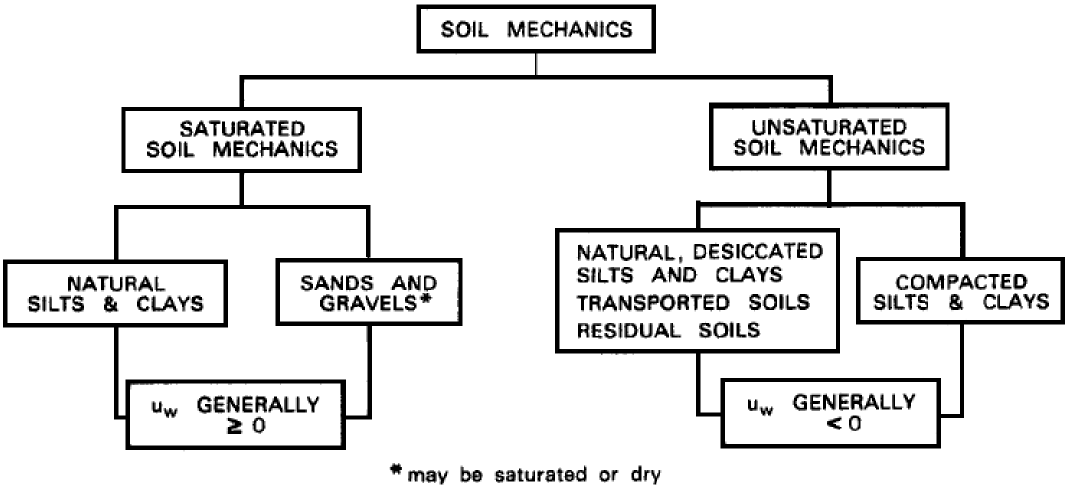


Figure 2.21 Classification of soil mechanics (Fredlund and Rahardjo, 1993)

Figure 2.22 shows that an unsaturated soil can be near to 100% saturation in the capillary zone and completely dry near the ground surface. The behavioural mechanics for an unsaturated soil have been primarily developed for the case, where the air and water phases are continuous (i.e., two-phase zone). The degree of saturation for the two-phase zone generally ranges from about 20-80%. However, it has been found that the proposed theories can be extended throughout the entire unsaturated soil spectrum (Fredlund and Rahardjo, 1993). The difference between saturated and unsaturated soil based on stress variables is shown in Figure 2.23.

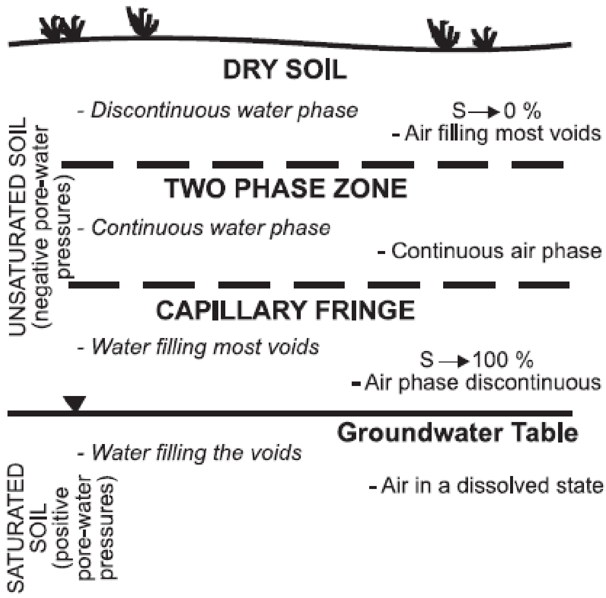


Figure 2.22 Classification of the regions within a saturated-unsaturated soil profile (Fredlund, 2000)

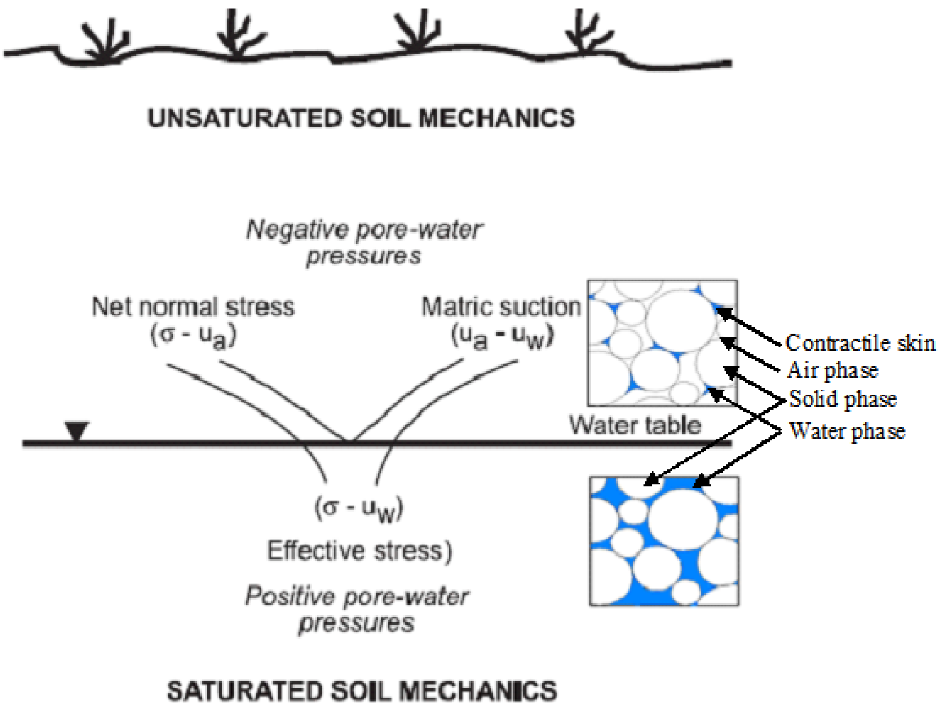


Figure 2.23 Difference between saturated and unsaturated soil mechanics based on the stress state description (after Fredlund, 2000).

Barden et al. (1973) reported that collapse in a soil, whether it is a sand, a silt or a clay, is related to its open metastable structure, which is of a bulky granular type. In the case of clays, the granular 'grains' are composed of aggregates of clay plates or 'packets'. The collapsible silt or sand grains also normally have clays as a dominant bonding agent, which clothes the surface of the grains. Where concentrated in a local area, the clays give a buttress type of support to the bulky grains. Burland (1965) and Burland and Ridley (1996) highlighted this, as shown in Figure 2.24. A highly idealised mechanistic model of a partly saturated silty-clay consists of silt grains and 'packets' of clay particles bonded together by high curvature menisci of water. Volume change can take place as a result of contact slip between grains and/or 'packets' as well as by shearing, swelling and shrinkage of the 'packets' themselves. A conceptual model of particle arrangements was proposed by Collins and McGown (1974), is shown in Figure 2.25.

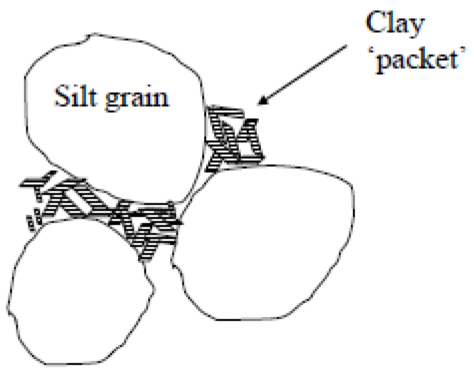


Figure 2.24 Idealised mechanistic model of a partly saturated silty clay (Burland and Ridley, 1996)

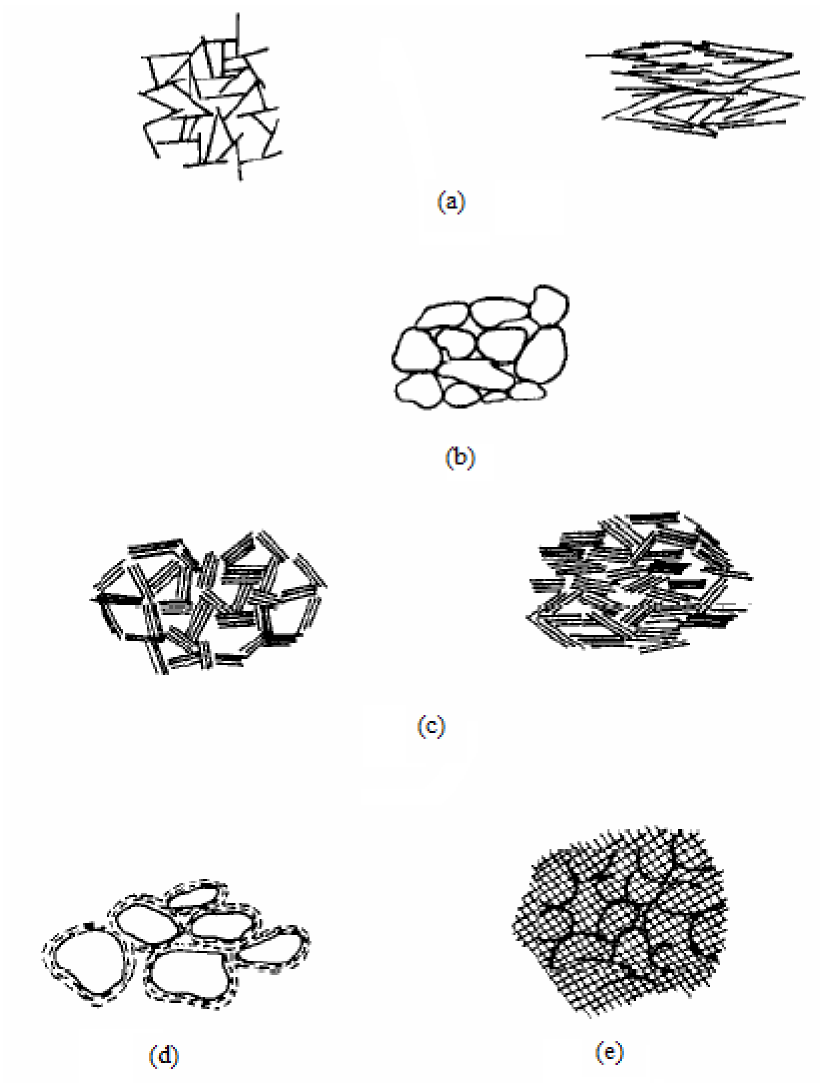


Figure 2.25 Schematic representation of elementary particle arrangements a) individual clay platelet interaction. b) individual silt or sand particle interaction. c) clay platelet group interaction. d) clothed silt or sand particle interaction. e) partly discernible particle interaction (Collins and McGown, 1974)

The details of soil micro-fabric in unsaturated soil are very important, because the micro-fabric controls the condition of the pore water, particularly, its negative pressure or suction. The types of soil micro-fabric for unsaturated compacted soil are shown in Figure 2.26. Within these forms of micro-fabric, there are three levels of particle arrangement which are: (i) elementary particle arrangements or quasi-crystals, (ii) particle assemblages and (iii) pore spaces. Gens and Alonso (1992) pointed out that the micro-fabric as shown in Figure 2.26 (a) is predominant in expansive soils or clayey soils compacted wet of optimum. Conversely, soil compacted at a water content dry of optimum could be viewed as a collection of saturated or nearly saturated aggregations, separated by relatively large pores at low saturation as shown in Figure 2.26 (b). In fabric *a*, the particle assemblages are formed by arrays of elementary particle arrangements and they are described as matrices. The pore space is made up of intra matrix pores existing between elementary particle arrangements. In fabric *b* the elementary particle arrangements join together to make aggregates resulting in a three-dimensional structure of a granular type. Both inter and intra-aggregate pore space exists. In both types of micro-fabric there is a further level of void space, the intra-element pores separating the clay platelets in the elementary particle arrangements (Figure 2.26c). If the clay minerals constituting the elementary particle arrangements belong to expansive types, micro-fabrics *a* and *b* will result in highly swelling soils, however, collapse phenomena can also occur in some circumstances, particularly, in the case of micro-fabric type *b*.

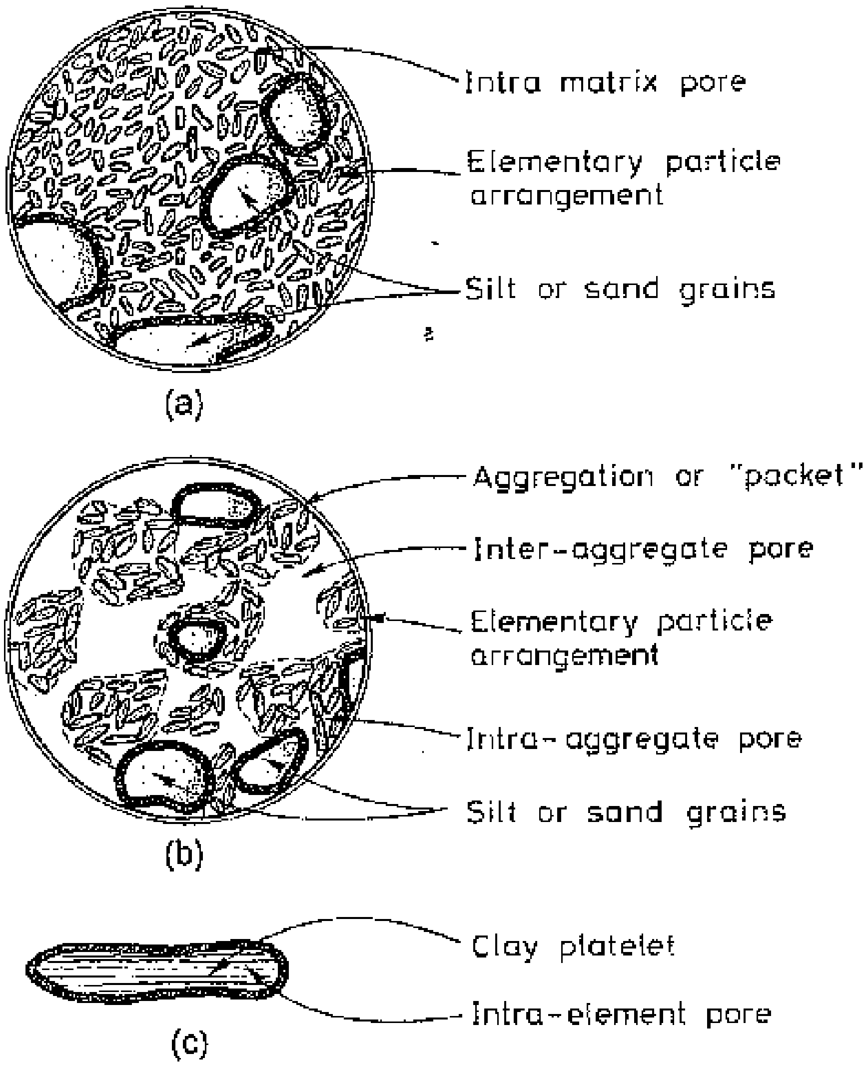


Figure 2.26 Fabric types in a compacted clay a) Clay matrix predominantly constituted by elementary arrangement of clay platelets, b) Micro-fabric of a clay predominantly made up of aggregations of elementary particle arrangements and (c) Elementary particle arrangement in parallel configuration (Gens and Alonso, 1992)

Compacted subgrade soil is considered as in an unsaturated state after sealing the ground surface (Fredlund and Rahardjo, 1993; Mancuso et al., 2002; Gupta et al., 2007; Yang et al., 2008; Murray and Sivakumar, 2010; Siekmeier, 2011; Ng and Xu, 2012). The mechanical behaviour of a compacted soil is highly dependent on various parameters including the volume mass soil properties, void ratio, gravimetric water content, soil suction and the degree of saturation (Pereira and Fredlund, 2000; Sun et al., 2010). Sun et al. (2007) conducted a series of triaxial tests on Pearl clay (a liquid limit of 49%, a plasticity index of 22 and a specific gravity 2.71). They reported that the degree of saturation changes more gradually with decreasing suction (unlike the volumetric strain), regardless of the mean net stress values and the initial density (Figure 2.27). The soil-water characteristics' curve is dependent on the soil density and is indirectly dependent on the stress state.

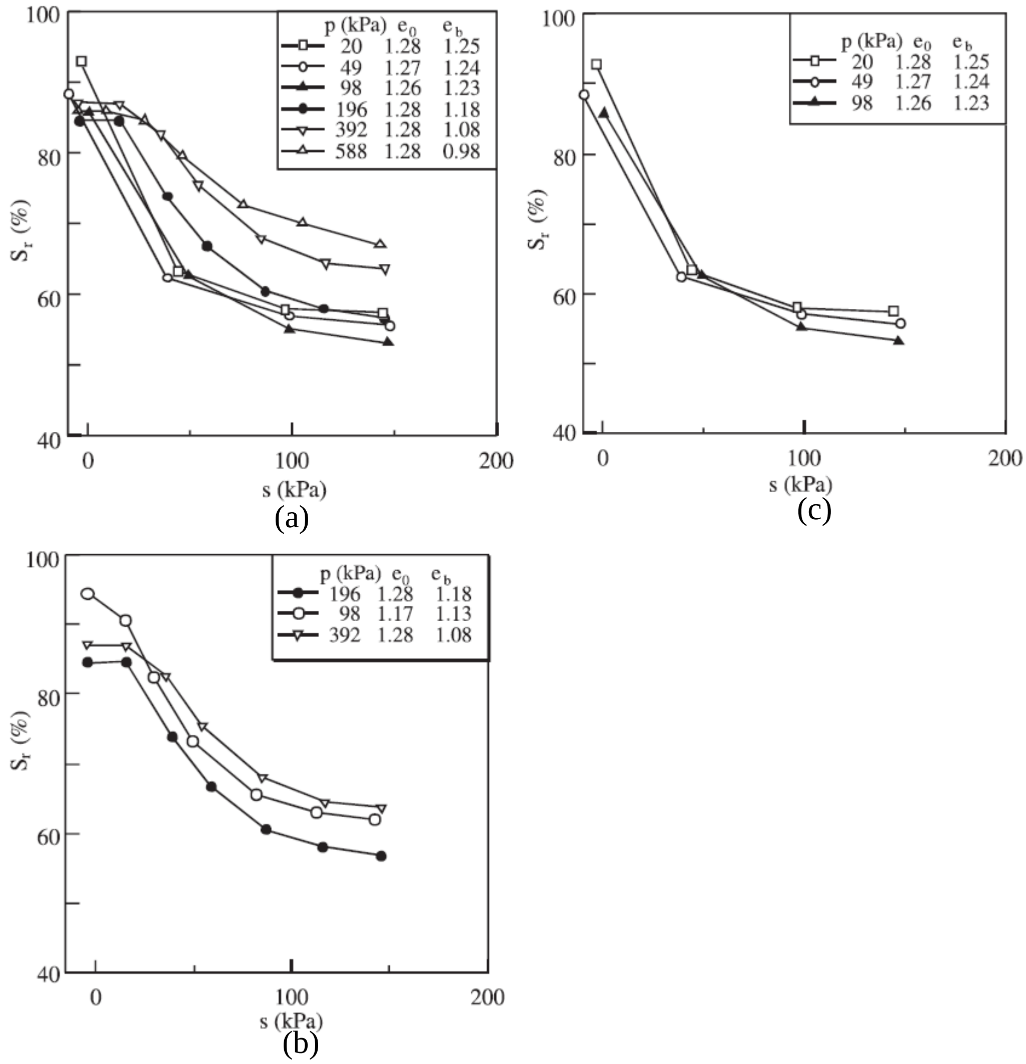


Figure 2.27 Soil–water characteristics during suction reduction but different stress states (a) at similar densities (b) different void ratio and (c) same void ratio (Sun et al., 2007)

Conventional soil mechanics considers soil in the dry condition or fully saturated conditions. Shear strength of soil has been modelled using Terzaghi's equation in which both strength and volume changes are controlled by the effective stress (Blight, 2013). For a saturated soil, the effective stress equation is:

$$\sigma' = \sigma - u_w \qquad (2.13)$$

Where, σ' is the effective stress, σ is the total stress and u_w is the pore water pressure. For a dry soil, the effective stress is:

$$\sigma' = \sigma - u_a \tag{2.14}$$

where u_a is the pore air pressure.

The principle of effective stress has been successfully applied in describing the mechanical behaviour of saturated soil but it cannot be applicable in unsaturated soil. Bishop (1959) extended the effective stress equation and introduced a new parameter χ, which is related to the degree of saturation.

$$\sigma' = (\sigma - u_a) + \chi(u_a - u_w) \tag{2.15}$$

where u_a is the pore pressure, u_w is the pore water pressure and σ is the total normal stress.

Jennings and Burland (1962) criticised the validity of Bishop's effective stress equation, as it cannot provide an explanation for partially saturated soil behaviour. From the experiments with silty soil, sample volume decreased during inundation at a constantly applied stress, even, when the effective stress decreases (Figure 2.28). The open points are for air-dried silt and the full points for identical samples soaked under zero stress. One of the air-dried samples was soaked under a constant load of 400kPa; subsequently, 'collapsed' onto the fully saturated line at A. For another sample, soaking took place while gradually reducing the vertical load to maintain an approximately constant void ratio. The second sample reached its saturation at point B. According to the principle of effective stress, as the sample is soaked, resulting in a reduction in suction and effective stress, the samples would have been expected to swell. On the contrary, the first sample experienced 'collapse' to point A and the void ratio of the second sample remained unchanged at B.

To overcome this problem, Bishop and Blight (1963) suggested that net stress and matric suction must both be considered as an independent stress variables. It is now agreed that unsaturated soil behaviour is governed by the two stress state variables which are net normal stress and matric suction (Bishop and Blight, 1963; Burland, 1965; Fredlund and

Morgenstern, 1977; Alonso et al., 1990; Houlsby, 1997; Gupta et al., 2007; Sawangsuriya et al., 2008; Ng and Xu, 2012).

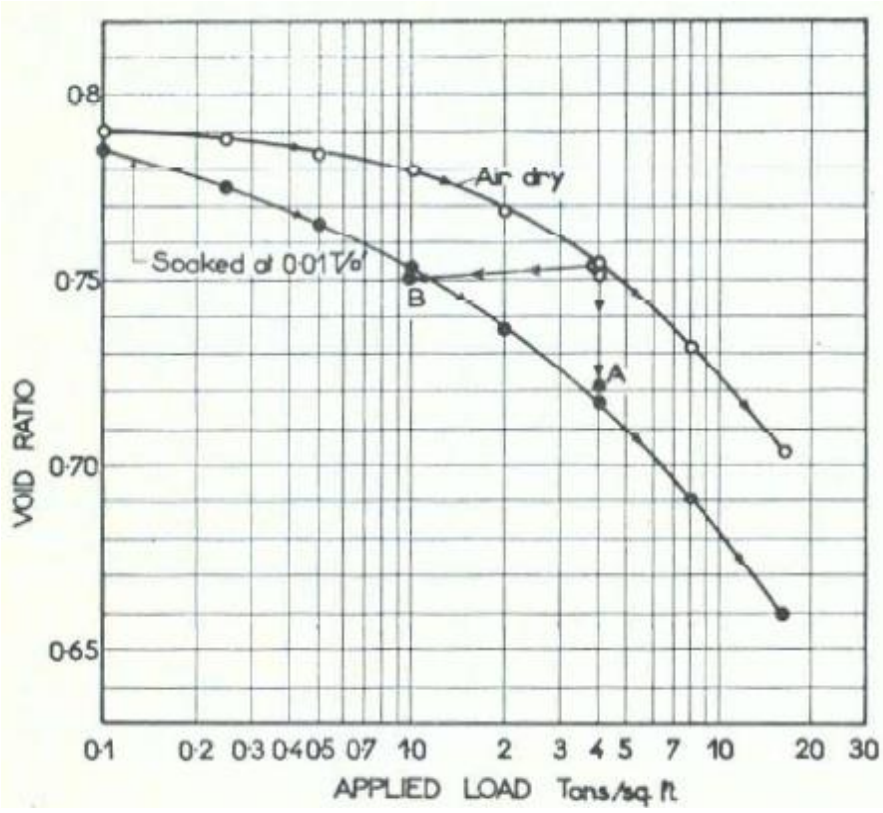


Figure 2.28 Oedometer compression curves on air dry (open points) and saturated (full points) silt showing the effects of soaking (Burland, 1965)

Guan et al. (2010) observed that the shear strength of soil behaviour in drying conditions, such as a hot and dry period, is different than during the wetting phase. The shear strength of soil changes because of the soil phase change from the dry to wet phase, hence the shear strength behaviour is different in different phases (Guan et al., 2010). The stiffness is greatly influenced by the state of stress and is also sensitive to the moisture and suction variations (Gupta et al., 2007). Gupta et al. (2007) performed a series of stiffness tests by triaxial on red wing-silt (RW), MnRoad-lean clay (MnRd), red lake falls-lean clay (RLF) and DI TH 23 slopes-fat clay (TH23). The resilient modulus based on external and internal displacement measurement as a function of the measured suction is shown in Figure 2.29. Subgrade modulus increases with increasing soil suction.

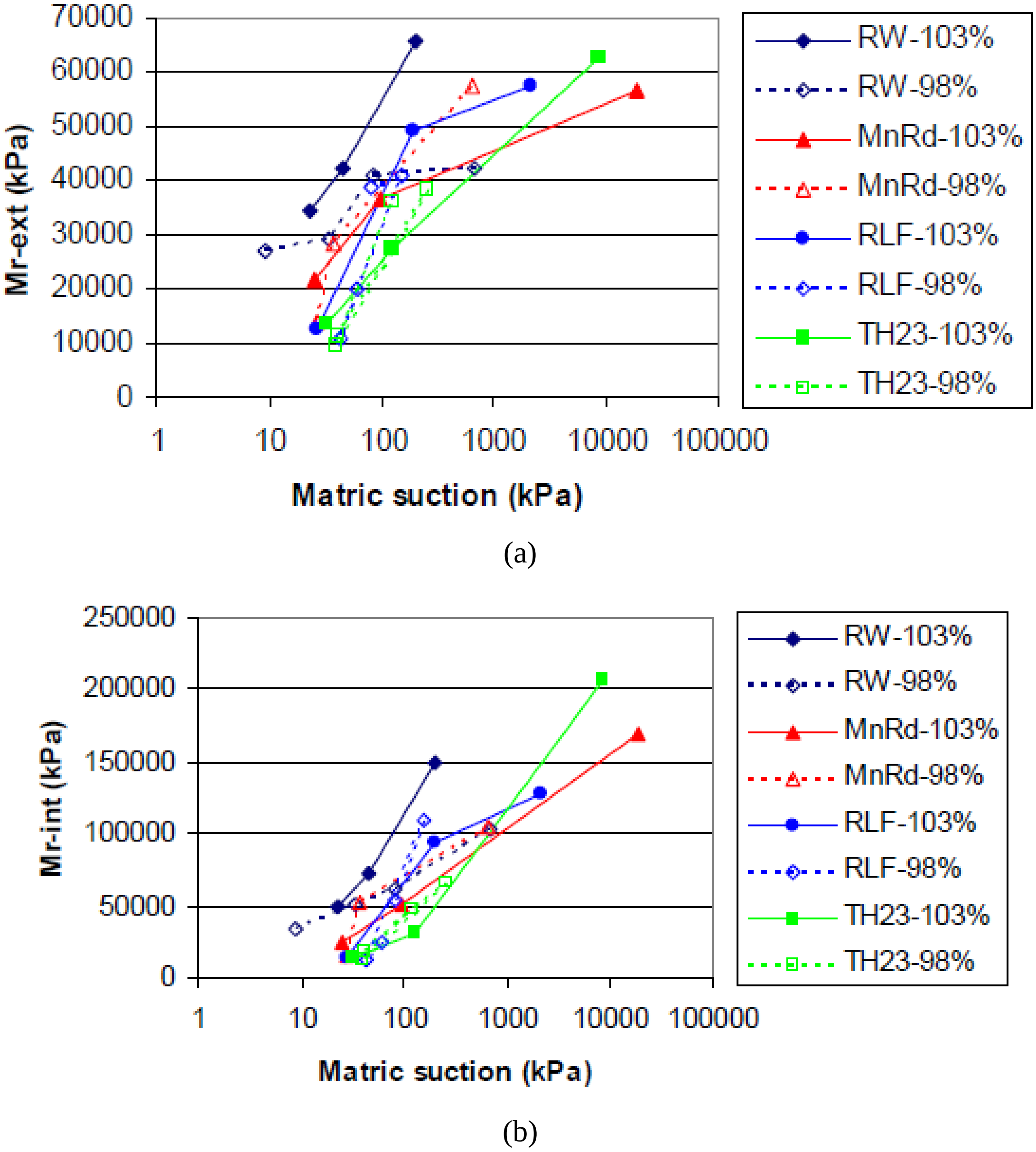


Figure 2.29 (a) Resilient Modulus (a) based on external displacement measurements (b) based on internal displacement measurements vs matric suction (Gupta et al., 2007)

Mancuso et al. (2002) explained the unsaturated soil behaviour by a simplified model based on the role of menisci water and bulk water (as shown in Figure 2.30). In the case of bulk water, the principles of saturated soil mechanics are likely to predict the soil's behaviour. The presence of a curved interface between particles only infers that the pore-water pressure is less than nearby air pressure. Therefore, changes in suction correspond to reductions of pore-water pressure. This can be characterised by a contact force (F) between the particles, which changes linearly with variations in matric suction (u_a-u_w). The linear behaviour continues until the air-entry value is crossed; the behaviour of soil moving from a saturated to an unsaturated condition can be explained by the increase of F along the bulk water curve (Figure 2.30):

$$F = \pi r^2 (u_a - u_w) \tag{2.16}$$

where r is the radius of the spherical particle, u_a is the pore air pressure and u_w is the pore water pressure. Beyond the air-entry value, the soil behaviour tends toward the menisci water curve, as defined in Figure 2.30, which explains the way in which a real soil moves from bulk water controlled behaviour to menisci water dominated behaviour. Soils are particulate materials; therefore, the properties of soil are governed by interparticle forces. Han et al. (1995) reported that the interparticle forces in the soil depend on the difference between the pore-air and pore-water pressures (matric suction) and their contact area with the soil particles. In the wetting path, soil having a low moisture content will have a smaller area of contact between the soil particles; whereas, in the drying path, soil will have a high moisture content and a larger area of contact. As a result, the interparticle forces in soil at wetting will be smaller than the drying path. In unsaturated soils, the negative pore-water pressure in menisci at particle contacts increases the interparticle forces, changes the small-strain stiffness, and alters the soil strength (Cho and Santamarina, 2001; Gupta et al., 2007).

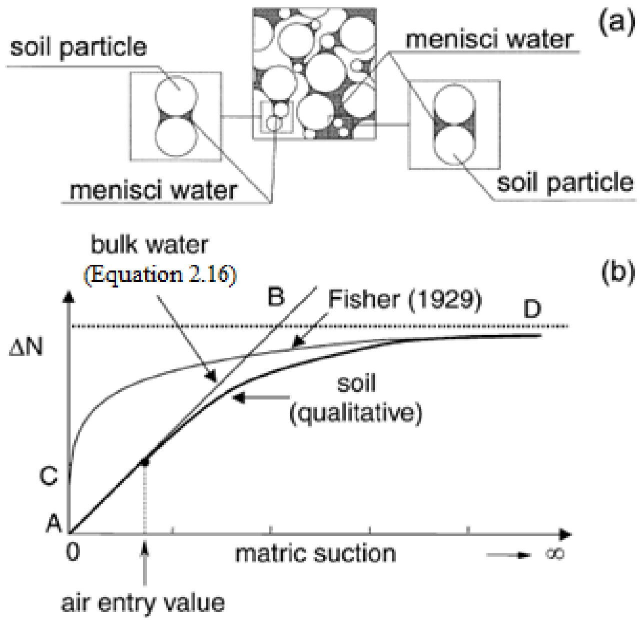
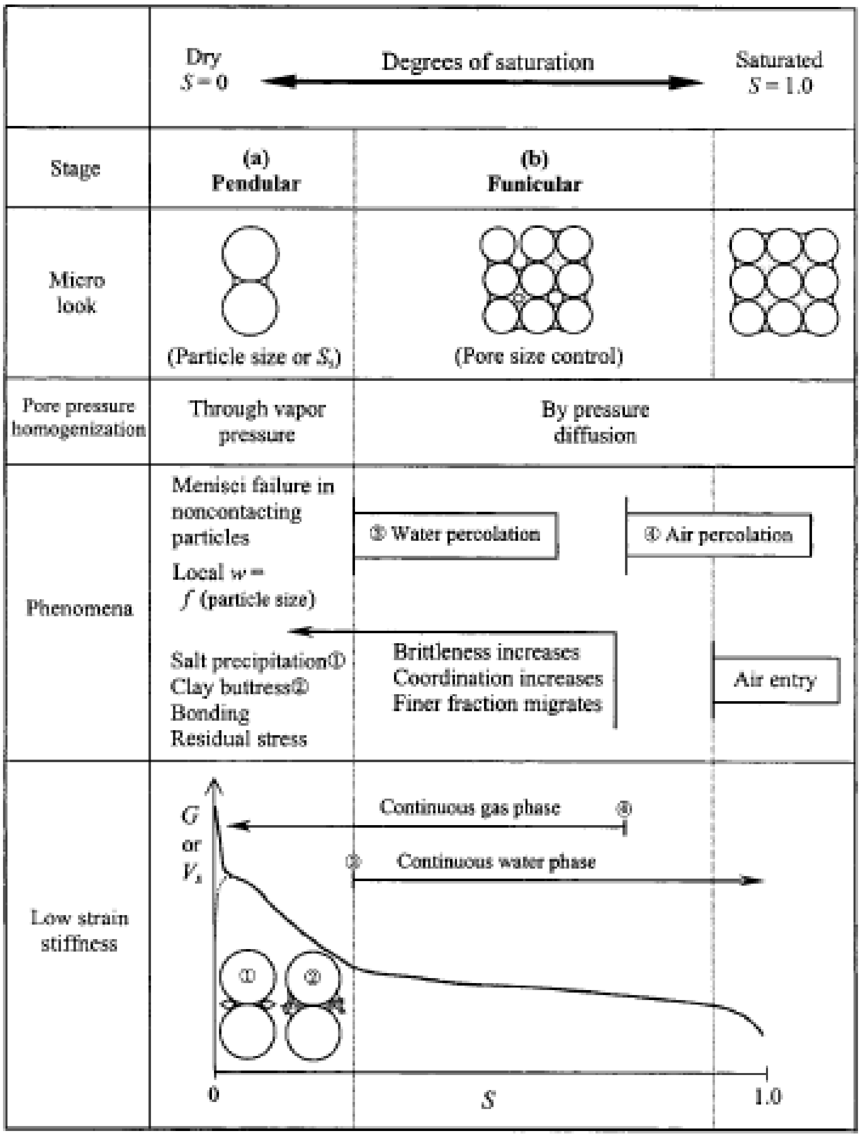


Figure 2.30 Influence of suction on the normal force (F) between two spherical soil particles for menisci and bulk water (after Mancuso et al., 2002)

Cho and Santamarina (2001) clarified different stages of unsaturated states, which apply to most natural soils subjected to drying (Figure 2.31). According to Laplace's equation (2.18), when water starts drying or draining from a saturated soil the outer menisci at boundaries pull inward and suction pressure increases. At the beginning of drying, when the change in water content is very small, the change in pore pressure has a significant global effect on the soil mass which remains saturated away from the boundary. The air-entry value is that point when the air phase breaks through into the pore structure (Kohgo et al., 1993; Aubertin et al., 1998; Cho and Santamarina, 2001). The air-entry value depends on pore size; so the finer particles with smaller pore throats have higher air-entry values; generally, it happens when the degree of saturation lies between 90% and 100%. Once air breaks in, the soil mass becomes unsaturated, however, water still forms a continuous phase called the funicular stage (Newitt and Conway-jones, 1958; Leverson and Lohnes, 1995; Cho and Santamarina, 2001). As the drying process continues, the suction pressure increases slowly and decreases the degree of saturation, following a quasi-linear trend. Any local change in water pressure homogenised rapidly throughout the mass by pressure diffusion within a continuous water phase. When the water becomes disconnected the pendular stage begins (Cho and Santamarina, 2001). Water rings form around particle contacts, only an adsorbed film may be present on particle surfaces (Leverson and Lohnes, 1995). The suction pressure increases significantly as the radii of menisci are small and, as this is only a contact-level force, a change in suction within a meniscus is felt at other menisci through the corresponding change in vapour pressure (Cho and Santamarina, 2001) and hence the total suction is as expressed in Kelvin's equation (2.21).



(G is small-strain shear modulus and V_s is shear wave velocity)

Figure 2.31 Stages of unsaturated conditions and related phenomena (after Cho and Santamarina, 2001)

2.6.1 Concept of soil suction

Soil suction is defined as the potential difference between pore water and water outside the soil pores (per volume of water); it is also referred to as the free energy state of soil water (Fredlund and Rahardjo, 1993). Most of the research associated with the pore fluid in a soil was initiated by soil physicists and agronomists during the late 1800's and probably the first group Corney et al. (1948, 1950) recognised the importance of soil suction in civil engineering (Krahn and Fredlund, 1972). In the soil mechanics symposium, "Moisture Equilibria and Moisture Changes in Soils' the subdivision of soil suction and the definitions quoted by the International Society of Soil Science were adopted (Aitchison, 1965 in Krahn and Fredlund, 1972).

Soil suction is a crucial stress parameter to understand the mechanical and hydraulic behaviour of unsaturated soil and it is essential for predicting the changing behaviour of unsaturated soil (Tarantino et al., 2011). Toll (1990) reported that the degree of saturation has a strong influence in the relationship between total stress, shear strength and suction. Suction influences the mechanical behaviour of unsaturated soil in two different ways (Wheeler and Karube, 1995):

i. It modifies the skeleton stress through changes in the average fluid pressure acting in the soil pores; and
ii. It provides an additional bonding force between particles contacts, often attributed to capillary phenomena occurring in the water menisci.

Soil suction is generally described as the potential of the soil for water attraction. Soil suction is a quantity that may be used to characterise the influence of moisture on volume, and is a measurement of the energy or stress that holds the soil water in the pores (Lytton et al., 2006). The soil suction, as measured in terms of the relative humidity, is called total suction. The total suction has two components, which are matric suction and osmotic suction. In most engineering problems, matric suction governs the unsaturated soil behaviour (mechanical problems); the osmotic suction is not commonly determined and is relatively insignificant (Fredlund and Rahardjo, 1993; Khoury et al., 2003; Gupta et al., 2007).

2.6.1.1 Matric suction

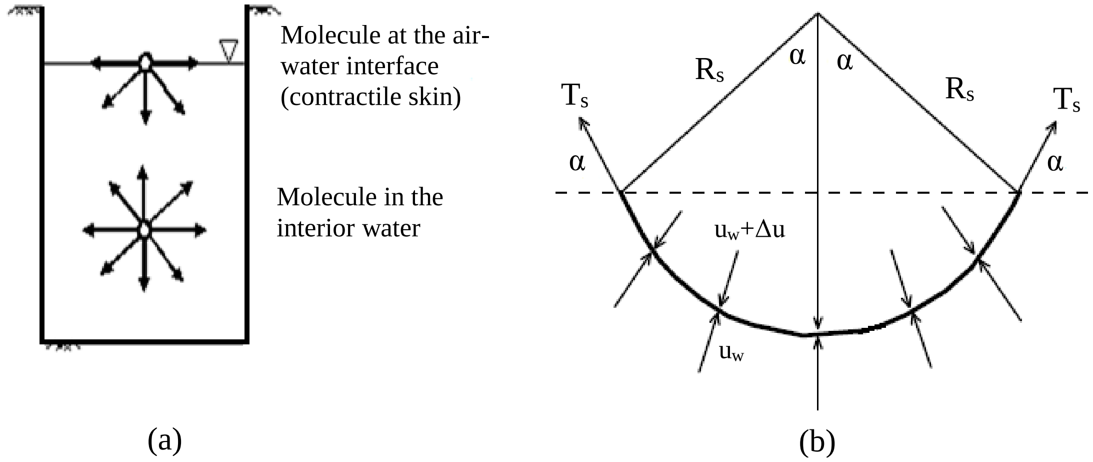
Matric suction is defined as a measure of the energy required to isolate a unit volume of pure water to overcome the attractive forces of water molecules and the attraction of water to solid surfaces (Cary and Zapata, 2011). Fredlund and Rahardjo (1993) stated the matric suction component is associated with the capillary phenomenon arising from the surface tension of water. Usually, the matric suction (ψ_m) is quantified as the difference between the pore air pressure u_a and the pore water pressure u_w which can be written as:

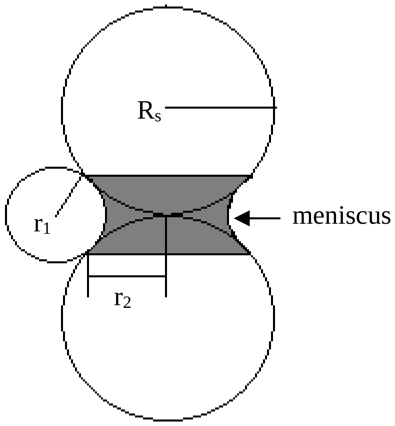
$$\psi_\mathrm{m} = (u_a - u_w) \tag{2.17}$$

Water in the soil pores is held by the potential energy of the tensile forces created because of curved interfaces and surface adsorptive forces (Gupta et al., 2007). In unsaturated soil, surface tension at air-water interface causes negative water pressure. Figure 2.32 presents the physical mechanisms of matric suction. The air-water interface, often referred to as the contractile skin possesses a property called surface tension, which is the results of inter-molecular forces which are different from those acting on molecules in the interior of the water (Figure 2.32a). The interior molecules of water experience equal forces in all directions. Whereas, a molecule at the air-water interface experiences an unbalanced force towards the interior of the water; to equilibrate the inward forces surface tension develops along the water surface. The contractile skin behaves like an elastic membrane due to the surface tension. In order to establish an equilibrium, a flexible two-dimensional membrane forms a concave curvature due to the different pressures on each side (Figure 2.32b). The two spherical particles diagram is shown in Figure 2.32c. The pressures on the membrane are u_w and $(u_w+\Delta u)$, radius of the curvature is R_s and surface tension T_s; according to Laplace equation:

$$\Delta u = (u_a - u_w) = T_s\left(\frac{1}{r_1} + \frac{1}{r2}\right) \tag{2.18}$$

where the pressure difference (u_a-u_w) is referred to as matric suction or the difference between pore air pressure u_a and pore water pressure u_w acting on the contractile skin.





(c)

Figure 2.32 (a) Surface tension phenomenon (b) Intermolecular forces at the air-water interface (c) pressures and surface tension acting on the curved two-dimensional surface (Fredlund and Rahardjo, 1993)

Matric suction is a result primarily of the phenomenon of capillarity, however, is also influenced by surface adsorption effects. The capillarity phenomenon is directly related to the surface tension of water and results, for example, in water rising up thin capillary tubes, as shown in Figure 2.33, and forming a curved surface between the water and air known as a meniscus. For equilibrium at the air-water (also known as the contractile skin) interface in the capillary tube, the pressure difference across the meniscus ($u_a - u_w$) is given by:

$$(u_a\text{-}u_w) = \frac{2T_c}{R} \tag{2.19}$$

where $(u_a - u_w) = \rho_w g h_c$, h_c is the capillary rise, R is the radius of curvature of the meniscus (where $R = r_t/ \cos\theta$), r_t is the radius of the capillary tube, T_c is the surface tension of the air-water interface (contractile skin) with units of force per unit length or energy per unit area and θ is the contact angle of the air–water interface with the wall of the capillary tube.

The pores in soil act as tortuous capillary tubes and result in the soil water rising above the water table. The finer the pores, the greater the meniscus curvature and the higher the water is elevated. The capillary water has a negative water pressure with respect to the air pressure and its magnitude is inversely proportional to the radius of curvature of the meniscus. In other words, negative pore water pressure, or matric suction, increases as the radius of the meniscus decreases. Consequently, fine-grained soils normally experience greater capillary rise than coarse-grained soils where the pore spaces are larger, though there are recognisable effects on the overall capillary rise from changes in pore diameter within soils, as soils comprise discrete particles not uniform tubes.

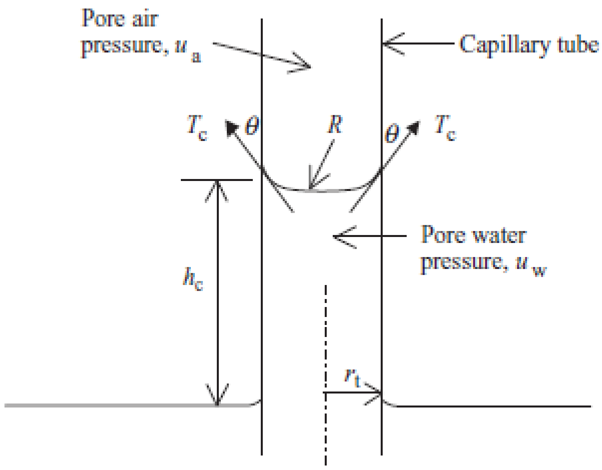


Figure 2.33 Capillary model (Murray and Sivakumar, 2010)

Lytton et al. (2006) reported that matric suction is an independent stress state variable which influences the total head for flow and the hydraulic conductivity of the unsaturated soil. The net normal stress (σ-u_a) and the matric suction (u_a-u_w) are the most important state variables to understand the mechanical unsaturated soil's behaviour, any changes in stress state variables are considered as deformation variables such as changing of void ratio, water content or degree of saturation (Fredlund, 2000; Mancuso et al., 2002).

2.6.1.2 Osmotic suction

Osmotic suction (π) is related to the presence of solutes in the pore water. Osmotic suction is generally considered to be present in unsaturated soil rather than saturated soil. However, the role of osmotic suction can be applicable in both saturated and unsaturated soil. Fredlund and Rahardjo (1993) reported an increment of dissolved salts in the pore water of the soil decreases the relative humidity which is referred to as osmotic suction (π). However, dissolved salts do not influence capillary phenomena, hence matric suction remains unchanged (Leong et al., 2007). Matric and osmotic suction are considered independent variables.

2.6.1.3 Total suction

The total suction (ψ) is expressed as a positive quantity and is defined as the sum of matric and osmotic suction. Fredlund and Rahardjo (1993) defined the total suction as the free energy of soil water and the matric and osmotic suction are the components of the free energy. The total suction can be written:

$$\psi = (u_a - u_w) + \pi \tag{2.20}$$

where ψ is the total suction, $(u_a - u_w)$ is matric suction, u_a and u_w are pore air and water pressure respectively and π is the osmotic suction.

Total suction is related to the relative humidity at the given temperature by the Kelvin's equations:

$$\psi = \frac{RT_s}{v} ln\left(\frac{P}{P_0}\right) \tag{2.21}$$

where ψ is total suction, R is the universal gas constant (8.314 $mol^{-1}K^{-1}$), T_s is the absolute temperature, v is molecular volume of water vapour (0.01802 m^3mol^{-1}), P is the partial vapour pressure of pore water (kPa) and P_0 is saturation vapour pressure of water at the same temperature (kPa).

Fredlund and Rahardjo (1993) reported that the soil behaviour changes with environmental changes, as well as changes of applied load; therefore, water content changes which is initially related to the matric suction of compacted soil. Osmotic suction usually does not influence on changes of water content. However, any changes in total suction may be changed by either one or both matric and osmotic suction. Matric suction is associated with the contractile skin, whereas osmotic suction is more associated with the diffuse double layer around the clay particles (Fredlund and Rahardjo, 1993). Krahn and Fredlund (1972) showed that matric and osmotic suctions are equivalent to the total suction (Figure 2.34).

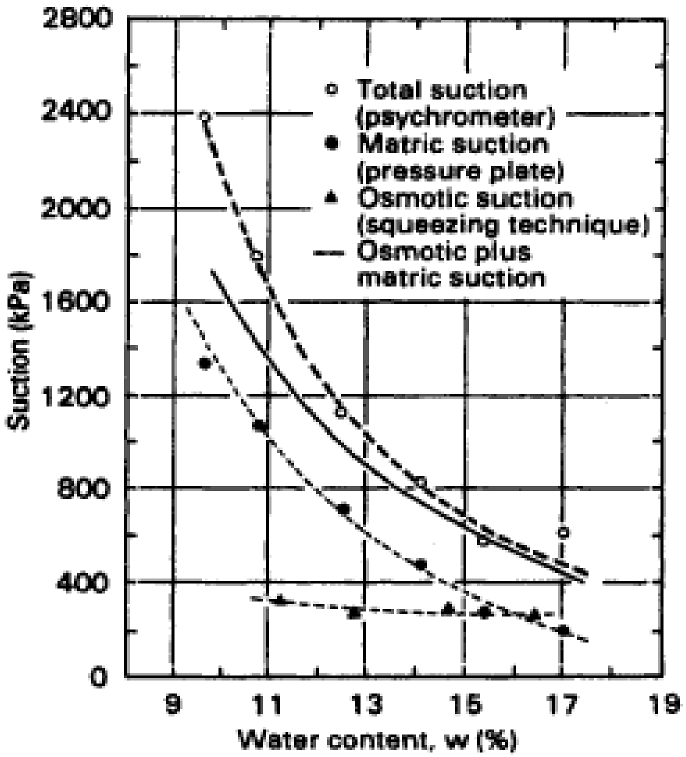


Figure 2.34 Total, matric and osmotic suction (Fredlund and Rahardjo, 1993)

2.6.2 Suction measurement

Suction measurement is an important factor in geotechnical analysis and design. For example, it is relevant to the issue of ground movement which occurs due to cyclic drying and wetting, where drying induces shrinkage and wetting induces swelling or collapse compression (Navaneethan et al., 2005). Tarantino et al. (2011) stated that suction measurement is a crucial factor to predict the mechanical and hydraulic behaviour of unsaturated soil. Generally, soil suction measurement can be divided into two categories: (i) direct measurement and (ii) indirect measurement (Fredlund and Rahardjo, 1993; Tarantino and Mongiovì, 2001).

2.6.2.1 Direct measurement

Direct measurement of suction involves intimate contact between the measuring device and soil. A tensiometer is commonly used to measure suction directly and a tensiometer may also be defined as a piezometer, which is specially designed to measure negative pore pressure (Ridley et al., 2003). It generally consists of a saturated porous ceramic filter (a high air-entry interface), a water reservoir and a pressure measuring device. The tensiometer allows water to be extracted from the reservoir into the soil across the porous filter. Water will flow between soil and reservoir through the filter until it reaches in equilibrium at a negative pressure between the water in the soil and in the reservoir. Subsequently soil suction can be measured by an electric transducer. If the tensiometer and soil make a good contact then matric suction can be measured. The limitation of this device is that it is only able to measure suction accurately up to 80kPa (Marinho et al., 2008). As suction increases, cavitation problems prevent effective suction transmission to the measuring device. Ridley and Burland (1993) introduced a high capacity tensiometer (up to 1500kPa) which overcomes the cavitation problem. A schematic diagram is shown in Figure 2.35. The device consists a high air-entry (1500kPa) ceramic filter and a small water reservoir of about 3mm^3 volume.

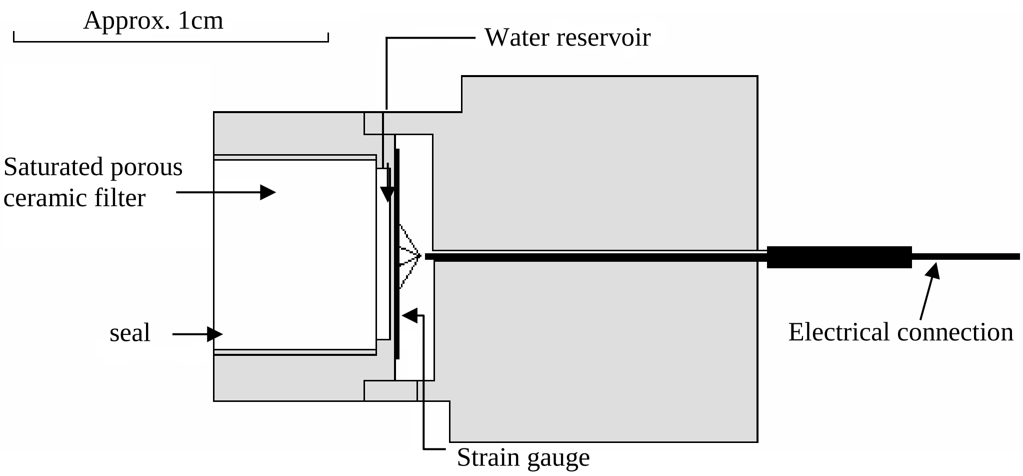


Figure 2.35 A schematic diagram of Imperial College tensiometer (Ridely and Burland, 1999)

2.6.2.2 Indirect measurement

Indirect measurement involves the measurement of, for example relative humidity, conductivity, moisture content which are then related to the suction through a calibration relationship (Tarantino and Mongiovì, 2001). Common techniques include electrical conductivity sensors, thermal conductivity sensors and the filter paper method (Guan, 1996). Bulut and Leong (2008) described how indirect suction measurement techniques can be determined by three different means: as primary (i.e. psychrometer) if it measures vapour pressure; secondary means it measures equilibrium condition through another porous medium (i.e. filter paper) and tertiary if the quantity measures other physical properties of the porous medium's moisture equilibrium condition (i.e. thermal and electrical conductivity sensors)

Suction measurement techniques are complicated due to certain limitations. Bulut and Leong (2008) pointed out two main sources that cause difficulties in the measurement of total suction by primary methods. Firstly, a small amount of change of relative humidity in the soil-air phase can lead to a significant change in suction. Secondly, minor temperature fluctuations may lead to a substantial change in suction. This primary method is not suitable to measure suction below 100kPa but it can measure up to 30MPa (Bulut and Leong, 2008). The limitation of the tertiary method (thermal conductivity sensor and electrical conductivity sensor) is defined by hysteresis.

Secondary method employing filter paper method is one of the most popular methods developed by Gardener (1937). This method is popular because of its low cost and wide range of suction measurement. This technique is based on the water absorption characteristics of a filter paper and has been adopted as a standard suction measurement method by ASTM D 5298. In this method, the filter paper is allowed to reach equilibrium with soil; either vapour for total suction, or liquid flow for matric suction. The suction of the soil can be measured from the water content of filter paper by using an appropriate calibration equation (Figure 2.36). The filter paper technique requires strict observance of the laboratory protocol otherwise it can interpret wrong results. Bulut and Leong (2008) reported that suction measurement by this method can be considered operator sensitive. However, this method is simple and affordable; it can measure almost from 0 to 30 MPa. Fredlund and Rahardjo (1993) also mentioned that using the filter paper method to determine the matric suction can be misleading as it measures both total suction and matric suction and depending on the contact between the filter paper and soil.

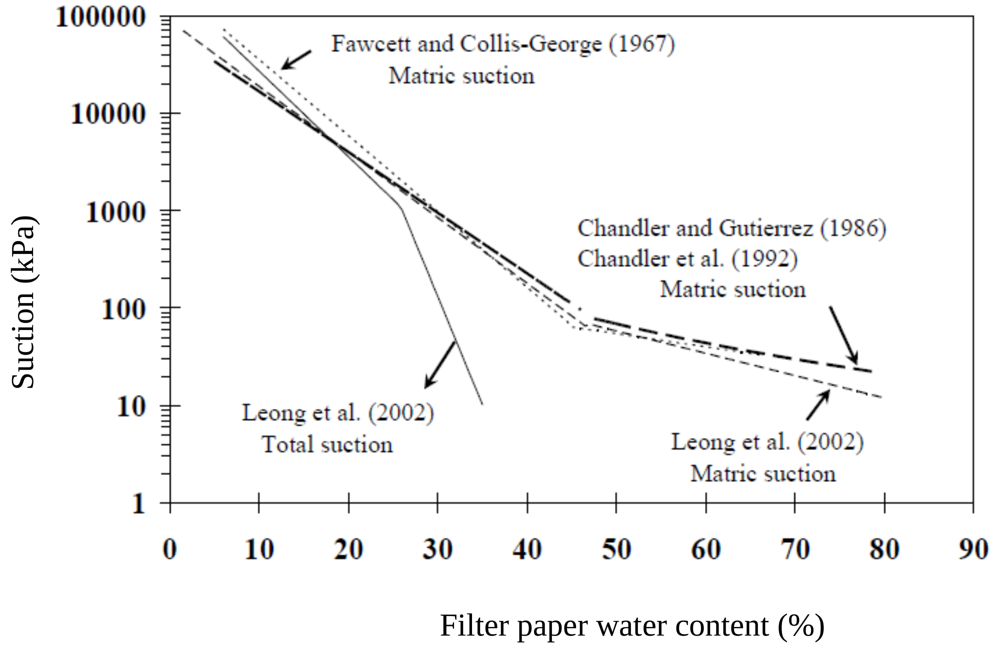


Figure 2.36 Filter paper calibration curves for total and matric suction measurements (Haghighi, 2011)

2.6.3 Suction control techniques

Three different types of system for controlling suction in the laboratory have commonly been used which are the axis translation technique, the osmotic technique and the vapour equilibrium technique.

2.6.3.1 The axis translation technique

The axis translation technique is one of the most commonly and widely used methods for imposing suction in the laboratory. This technique was first proposed by Hilf (1956) to control the suction values of the sample shown in Figure 2.37. The system is able to control air pressure and water pressure independently thus, suction can be imposed directly. The basic principle of this technique is to elevate the pore air pressure and pore water pressure by the same amount therefore the matric suction remains constant. Many researchers have used this technique to control the suction (Matyas and Radhakrishna, 1968; Rampino et al., 1999; Toll and Ong, 2003; Farouk et al., 2004; Estabragh and Javadi, 2008; Ng and Tse, 2008; Yang et al., 2008; Uchaipichat and Khalili, 2009; Kasangaki, 2012).

The maximum measurable suction by using an axis translation technique depends on the air pressure system, cell condition and the air entry value (AEV) of the porous stone. The

AEV value is important and must be higher than the matric suction in the soil sample, in order to avoid the entry of air bubbles in to the measuring system. Currently, the highest available air-entry value disk is 1500 kPa (Fredlund and Rahardjo, 1993; Marinho et al., 2008).

The limitation of this technique involves air-diffusion through the AEV porous stone in to the pore water pressure system. The formation of the air bubbles reduces the efficiency of this system, which can be overcome by a flushing system.

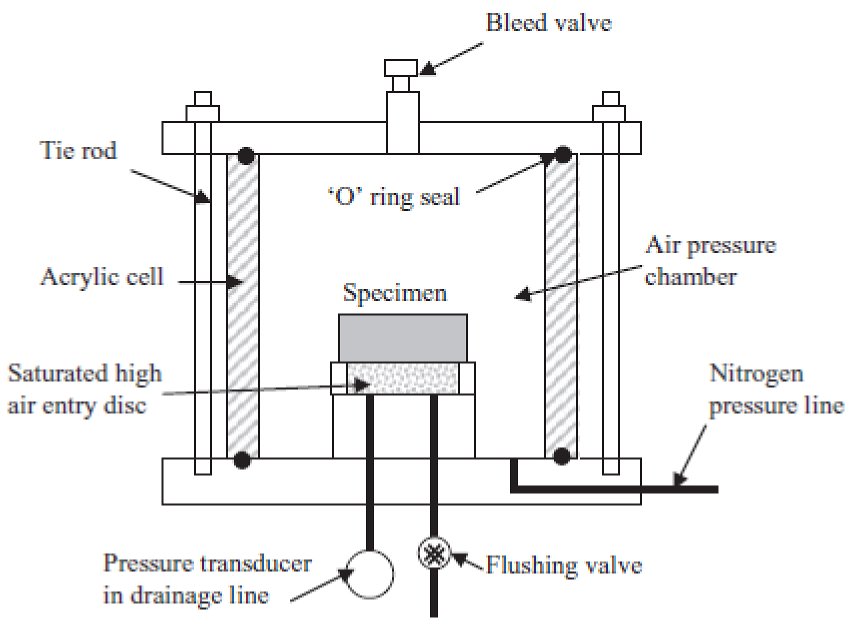


Figure 2.37 Axis translation technique device to measure matric suction (Murray and Sivakumar, 2010)

2.6.3.2 Osmotic technique

The osmotic technique was first used by Kassiff and Shalom (1971). The technique involves placing the sample in contact with a semi permeable membrane that separates the soil sample and a solution containing large size soluble polyethylene glycol molecules (PEG) that is circulated behind the membrane. The semi-permeable membrane only allows water to pass through but not PEG molecules, as a result osmotic suction is applied to the sample through the membrane. The osmotic technique is based on the principle of osmosis; the net movement of solvent through a semi-permeable membrane from a higher solute concentration to lower solute concentration, in order to equalise the solute concentration on the both sides. The imposed suction value depends on the concentration of the solution; the higher the concentration the higher the suction. Figure 2.38 presents the osmotic technique using triaxial test systems.

The advantage of this technique is that, it can reproduce real field conditions of soil suction as no artificial air pressure is applied to the specimen; therefore, this advantage can be significant for high degrees of saturation when the air continuity is no longer ensured (Delage et al., 2008). In this technique, a higher level of suction can be measured up to 10MPa as there are no air diffusion problems (Delage et al., 1998). One of the limitations of this technique is the degradation of the semi-permeable membrane over time; a condition which can reduce suction. The cellulose acetate membranes (most commonly used membrane) are highly sensitive to bacteria attack (Delage et al., 2008).

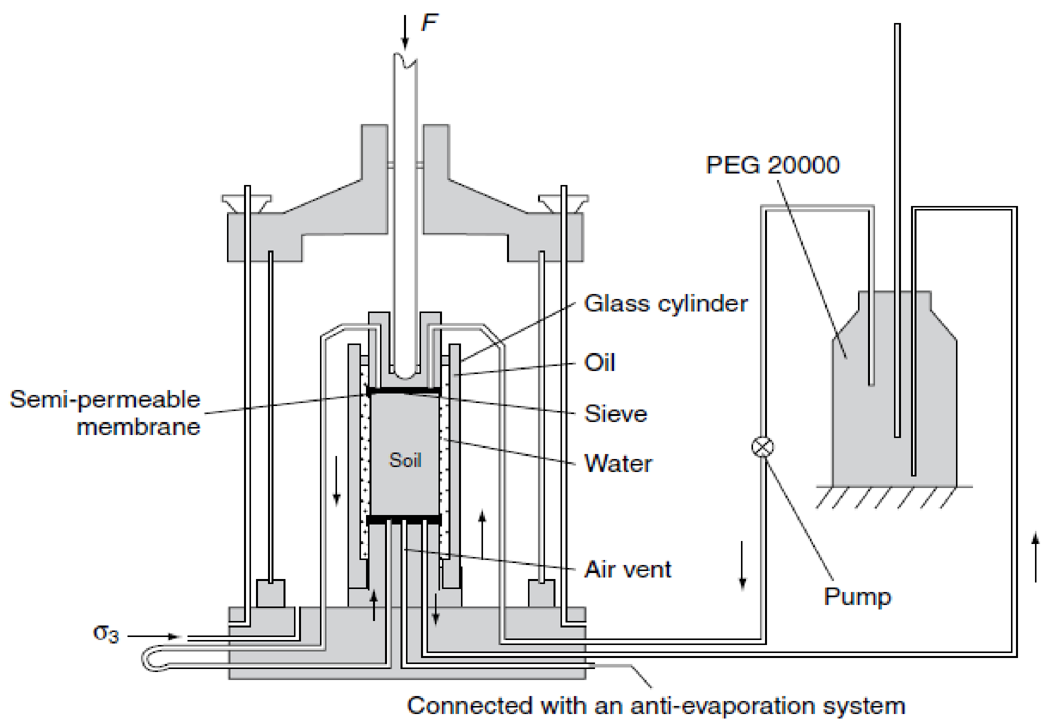


Figure 2.38 Triaxial systems for the osmotic technique (after Cui and Delage, 1996)

2.6.3.3 The vapour equilibrium technique

The vapour equilibrium, or relative humidity, technique is based on the control of the relative humidity of the air surrounding the specimen by the osmotic potential of chemical solution. This technique is implemented by creating an environment to control the relative humidity in a closed system, in which the sample is placed, as shown in Figure 2.39. Soil water potential is controlled by means of the movement of water molecules through the vapour phase from a reference system (which is put in a closed system) to the soil pores until equilibrium is reached (Delage et al., 2008). Three different types of chemical solutions used which are: (i) an unsaturated acid solution, (ii) a saturated salt solution and (iii) salt solutions at different concentrations. The salt solution inside the chamber cannot move through air from the reference system to the sample therefore only total suction is controlled. After achieving equilibrium, the relative humidity of the air inside the chamber is measured by either a psychrometer or hygrometer.

The main disadvantage of this technique is the required moisture equalisation time, it takes an extremely long time, up to several weeks (Delage et al., 2008). Another limitation mentioned by Delage et al. (2008), relating to the vapour equilibrium technique is maintaining temperature precisely between reference system and sample. This technique is not suitable for low suction (high humidity) (Romero et al., 2001). Delage et al. (1998) reported that the vapour equilibrium technique experienced some problems at suction levels below 10MPa. However, this technique is able to apply suction up to 150MPa.

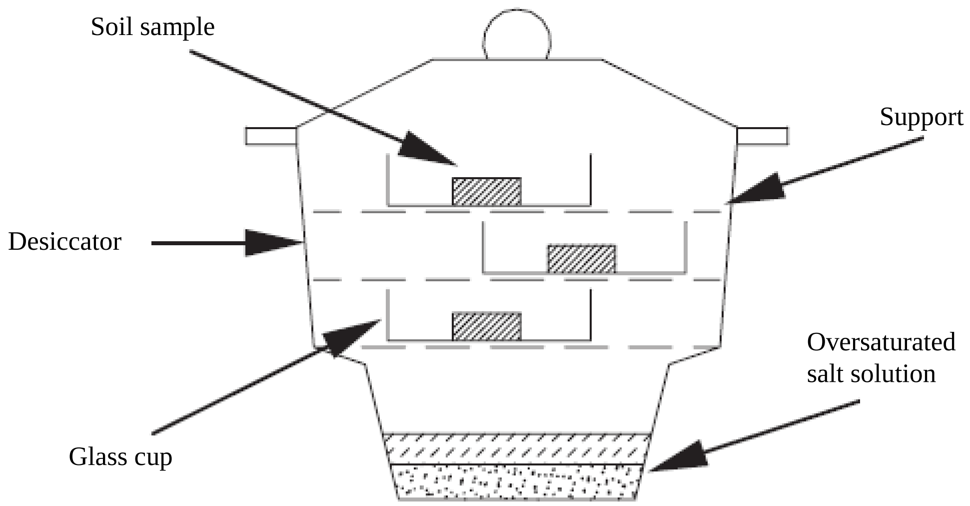


Figure 2.39 Imposing suction on three samples in a desiccator (Tang and Cui, 2005)

A summary of suction measuring techniques and their range of suction measurements are presented below table 2.4.

Table 2.4 Devices for measurement (approximate) ranges and times for equilibration in measurement and control of soil suction (after Murry and Sivakumar, 2010)

Instrument	Suction component measured	Typical measurement range (kPa)	Equilibration time
		Suction measurement	
Pressure plate	Matric	0-1,500	Several hours to days
Tensiometers and suction probes	Matric	0-1,500	Several minutes
Thermal conductivity sensors	Matric	1-1,500	Several hours to weeks
Electrical conductivity sensors	Matric	50-1,500	Several hours to weeks
Filter paper contact	Matric	0-10,000 or greater	2-57 days
Thermocouple psychrometers	Total	100-8,000	Several minutes to several hours
Transistor psychrometers	Total	100-70,000	About 1 hour
Chilled mirror psychrometer	Total	1-60,000	3-10 minutes
Filter paper non-contact	Total	1,000-10,000 or greater	2-14 days
Electrical conductivity of pore water extracted using pore fluid squeezer	Osmotic	Entire range	---
		Suction control	
Negative (or Hanging) water column technique	Matric	0-30 or greater with multiple columns or vacuum control	Several hours to days
Axis translation technique	Matric	0-1,500	Several hours to days
Osmotic technique	Matric	0-10,000	Up to 2 months
Vapour equilibrium	Total	4,000-600,000	1-2 months

2.7 Unsaturated soil behaviour under drying and wetting cycles

Soil properties change by repeated drying and wetting due to environmental influences. The repeated wetting and drying, resulting in the build-up or breakdown of soil particles and is also partly responsible for a continuous increase in the proportion of water stable aggregates (Allam and Sridharan, 1981). When the water content decreases by evaporation, the negative pore pressure increases and this suction force causes soil to shrink due to internal compression stresses; as a result, dry density increases (Dif and Bluemel, 1991). Sivakumar et al. (2006) reported from their experiments on compacted kaolin clay that the wetting and drying curves are significantly different is shown in Figure 2.40 (the atmospheric pressure (p_{atm}) of 100 kPa is introduced in the coordinate suction axis as $\ln[(s + p_{atm})/p_{atm}]$ to avoid difficulties of representing $s = 0$ when using a logarithmic scale (Alonso et al., 1990)). The difference is a result of hydraulic hysteresis and particles' volumetric and distortional changes. The volume change of unsaturated soil due to wetting and drying causes enormous damage in foundation (Aziz et al., 2006). Rao (2011) stated that the drying and wetting cycle can influence the swell and collapse performance of compacted soil. The cyclic drying and wetting can enhance soil stiffness and also increase the brittleness of soil. The drying phase has higher shear strength, lower stiffness, more ductility and contraction at shearing; on the other hand, the wetting phase has lower shear strength, high stiffness, more brittleness and dilation at shearing (Allam and Sridharan, 1981; Guan et al., 2010).

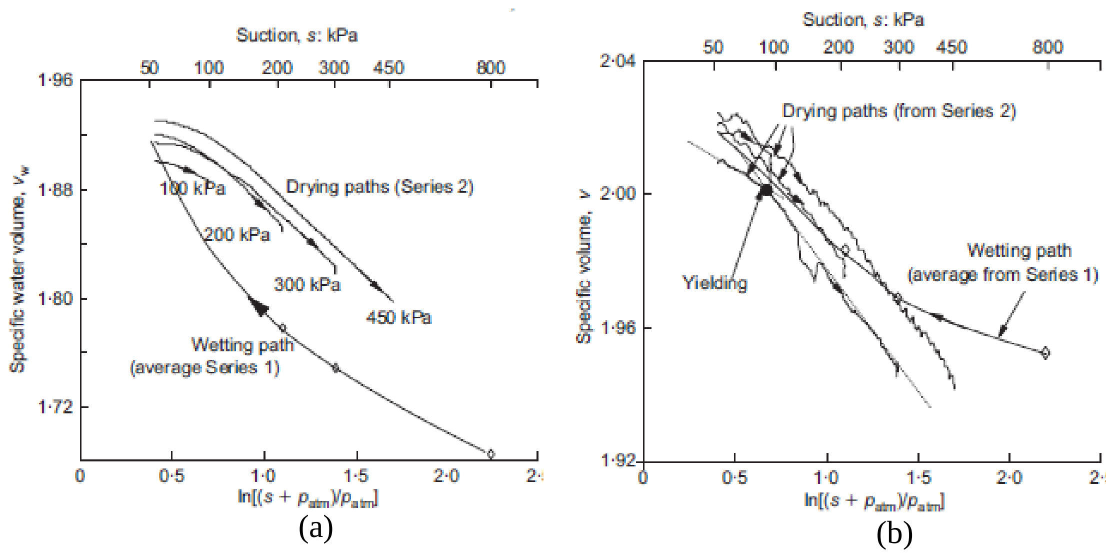


Figure 2.40 Suction characteristics during wetting and drying (specific water volume and (b) specific volume (Sivakumar et al., 2006)

Guan et al. (2010) conducted CD triaxial tests at different matric suctions on drying and wetting paths under a constant net confining pressure of 100 kPa. The cohesion intercept and shear strength of the specimen on the drying path appears to be higher than those of specimens on the wetting path at matric suction of 200 kPa is shown in Figure 2.41a. Figure. 2.41b and 41c, the total volume and water volume of both specimens (CD-100-1d50) and (CD-100-1d200) on the drying path decrease during shearing. On the other hand, the total volume of both specimens (CD-100-1w50) and (CD-100-1w200) on the wetting path increases during shearing. The water volume of specimen, (CD-100-1w200), decreases in the early stage of shearing and essentially there is no water volume change at the end of the shearing while the water volume of specimen, (CD-100- 1w50), decreases in the early stage of shearing and increases significantly in the later stage of shearing. From the experimental results, the sand-kaolin specimens under the drying process show less stiffness, more ductility, and contraction during the shearing stage, while the sand-kaolin specimens on the wetting path show more stiffness, more brittleness and dilation during the shearing stage. Therefore, the sand-kaolin specimens on the drying path are considered to be normally consolidated shearing behavior, while those on the wetting path are considered to be overconsolidated shearing behavior. (Goh et al., 2014) reported similar behaviour from their experimental study on sand-kaolin that the specimens on the first drying path have higher peak strength, lower stiffness and higher axial strain at failure and they also show more ductility and more contraction (or less dilation) during shearing than specimens on the first cycle wetting path at given matric suction (Figure 2.42). They concluded that the shearing behaviour on the first cycle drying path was similar to the shearing behaviour of a normally consolidated specimen and the first cycle wetting path specimens were like overconsolidated soil.

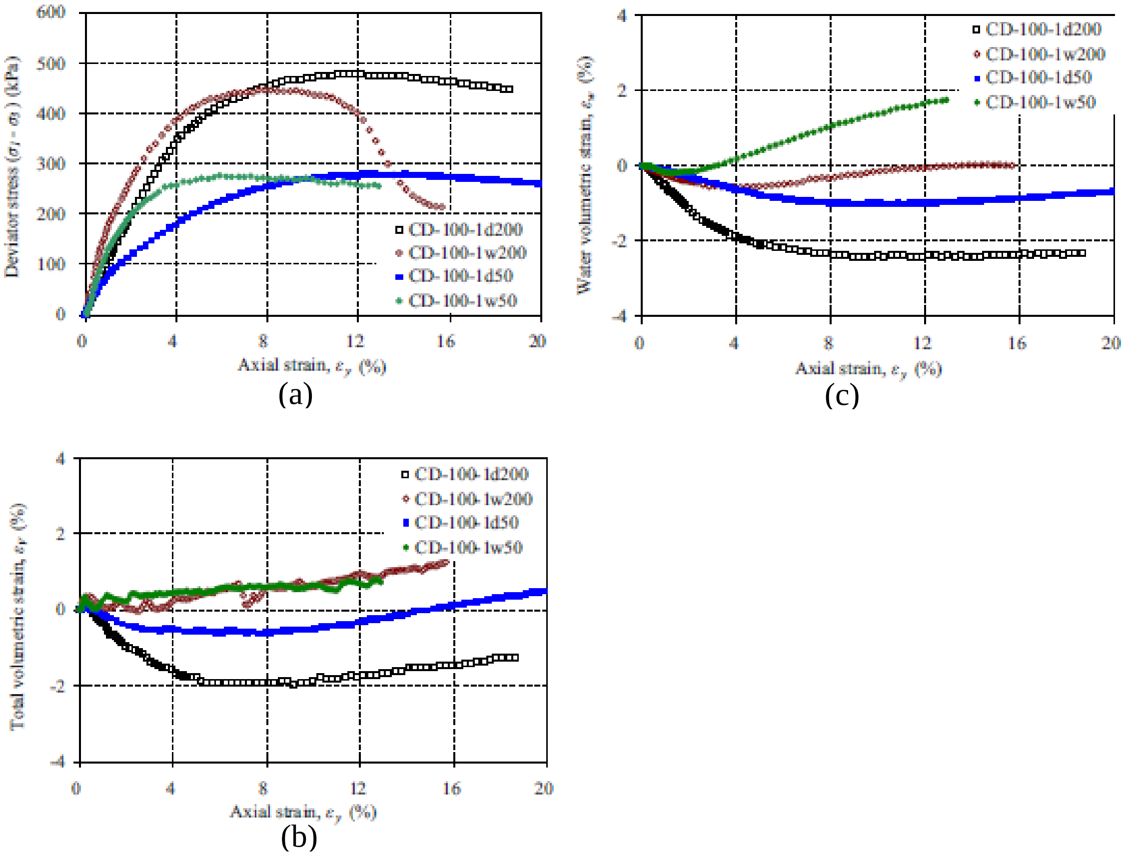


Figure 2.41 (a) Stress-strain relationships, (b) total volume change behaviour and (c) water content change behaviour of the sand-kaolin samples at matric suction of 50 and 200kPa on drying and wetting paths

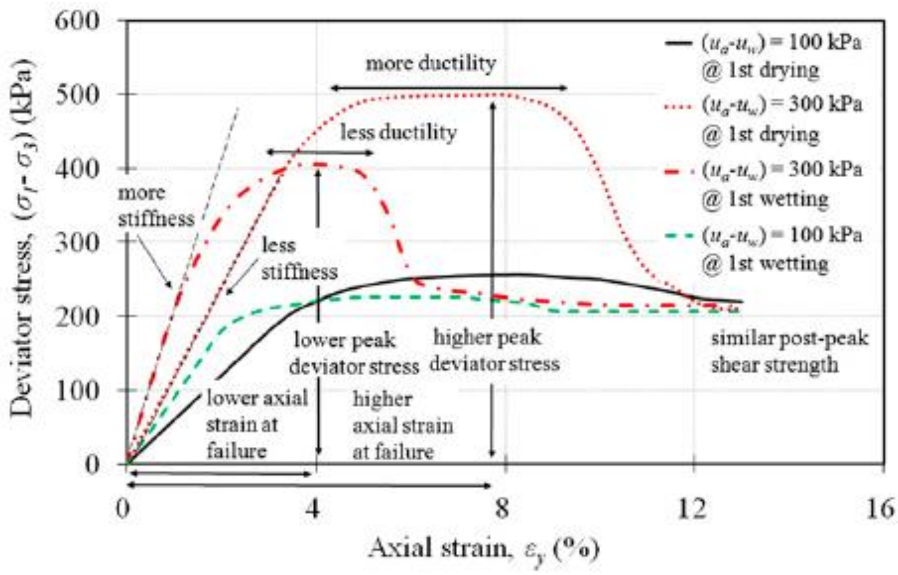


Figure 2.42 Stress-strain curves in the first cycle of drying and wetting (Goh et al., 2014)

When considering the shear strength behaviour of soil changes on wetting and drying cycles, it is widely accepted that the wet and dry cycles increase the shear strength of soil progressively. The normally consolidated soil turns into over-consolidated soil as the number of wetting and drying cycle increased (Allam and Sridharan, 1981; Kodikara et al., 1999). During the drying period, the water table found below the soil zone; the water flows from the deeper regions to near ground surface due to capillary action. The infiltration of water into the ground reduces the soil's strength properties due to loss of matric suction and experiences shear failure or swell/collapse strains under loading (Rao et al., 2011). Unsaturated soil behaviour is different in wetting and drying cycles due to hysteresis effect (Han et al., 1995; Gallage and Uchimura, 2006; Ng and Tse, 2008; Guan et al., 2010; Khalili and Zargarbashi, 2010; Goh et al., 2014). Figure 2.43 presents the hysteresis effect due to cycles of wetting and drying.

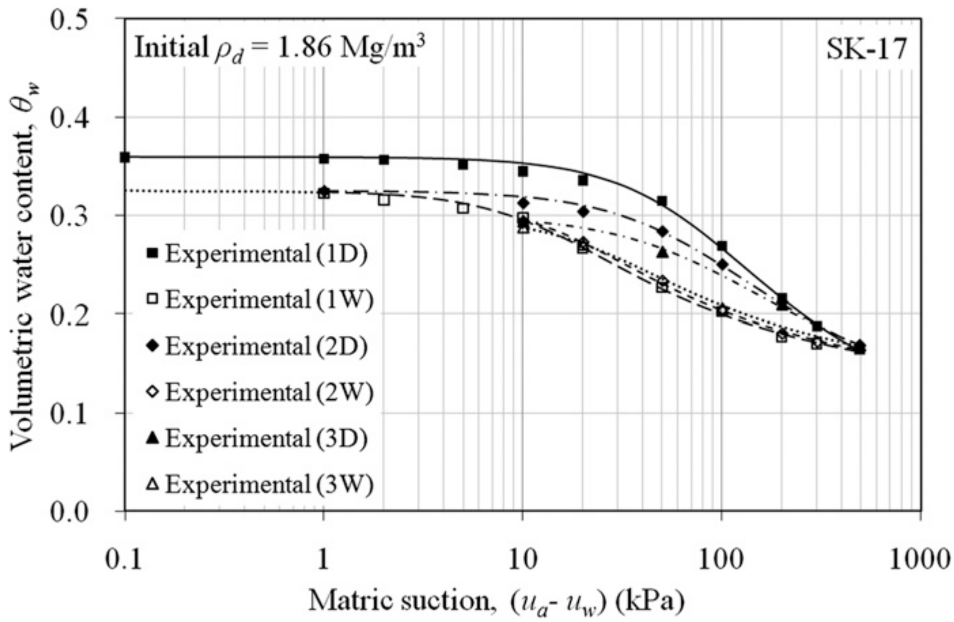
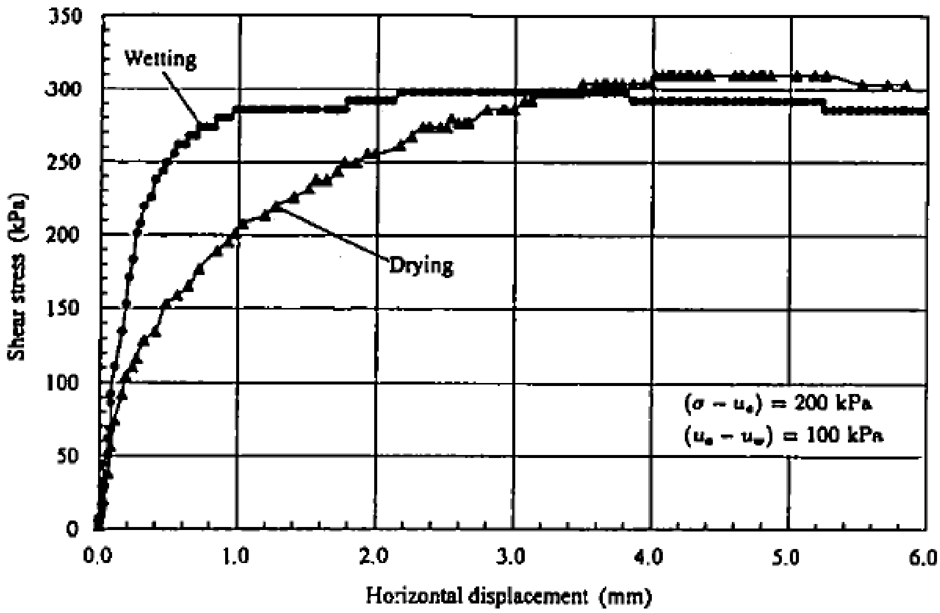
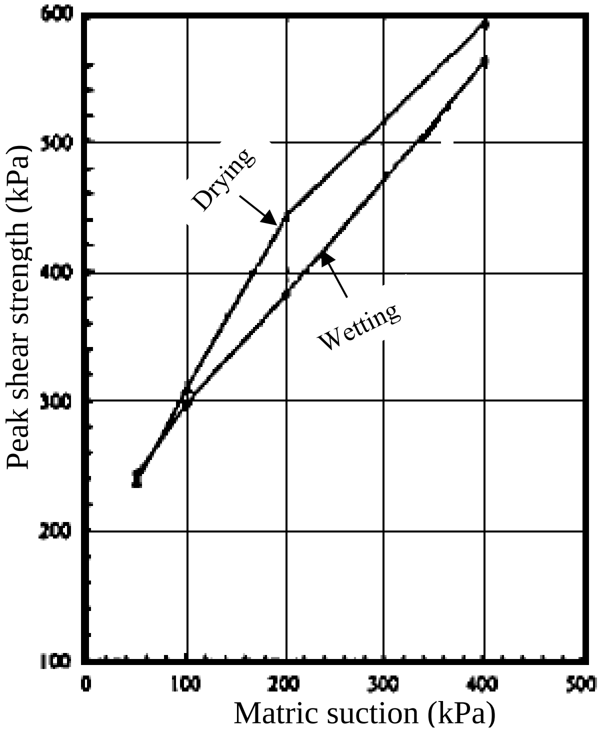


Figure 2.43 cycles of drying and wetting paths of sand-kaolin specimens under zero net confining pressure (1D denotes the first cycle of drying, 1W denotes, the first cycle of wetting and so on) (Goh et al., 2014)

Han et al. (1995) reported that the soil is stiffer and more brittle in wetting in its shear behaviour in comparison with the soil in drying. In the wetting, the brittle shear behaviour may explain the rapid instability of slope after rainfall infiltration. They also found that the shear strength in wetting is lower than that for the soil in drying (Figure 2.44). However, Gallage and Uchimura (2006) conducted a shear strength test (by modified triaxial apparatus) on Edosaki sand (16.4% on non-plastic fines) under low matric suction (50kPa), the findings contradicts the result of (Han et al., 1995). The specimen in the wetting process shows more contractive behaviour compared to the specimen in the drying process. It was observed that the shear stress in the wetting process is higher than that in the drying process under the same matric suction and net confining stress is shown in Figure 2.45. Nevertheless, most of the researcher agreed that matric suction contributes significantly to the shear strength of soil (Fredlund et al., 1978; Escario and Sáez, 1986; Alonso et al., 1990; Ng and Tse, 2008; Guan et al., 2010; Khalili and Zargarbashi, 2010; Sheng et al., 2011; Goh et al., 2014).



(a)



(b)

Figure 2.44 (a) Shear stress and versus horizontal displacement for specimens at drying and wetting (b) Peak shear strength versus matric suction at a normal stress of 200kPa (Han et al., 1995)

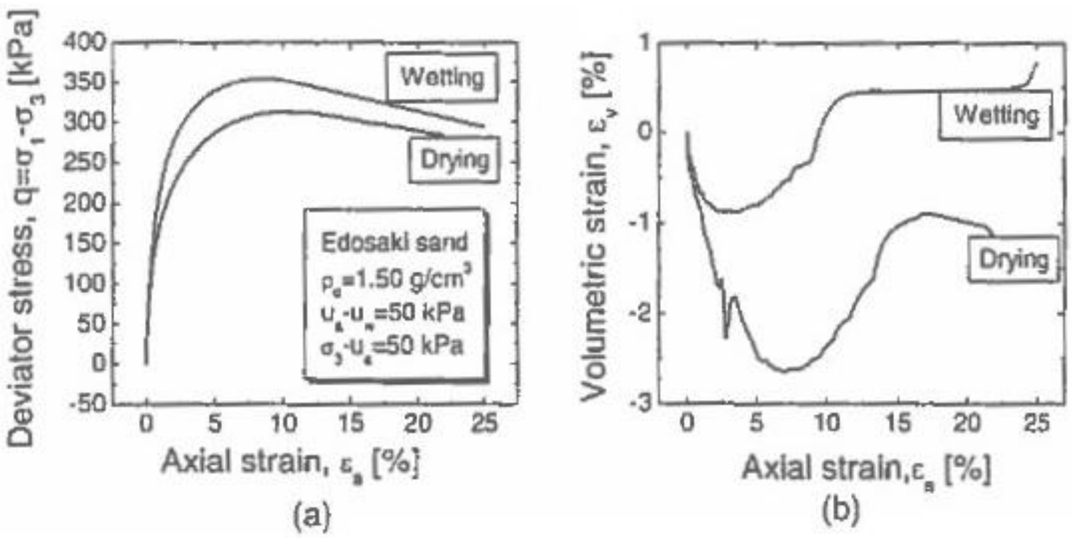
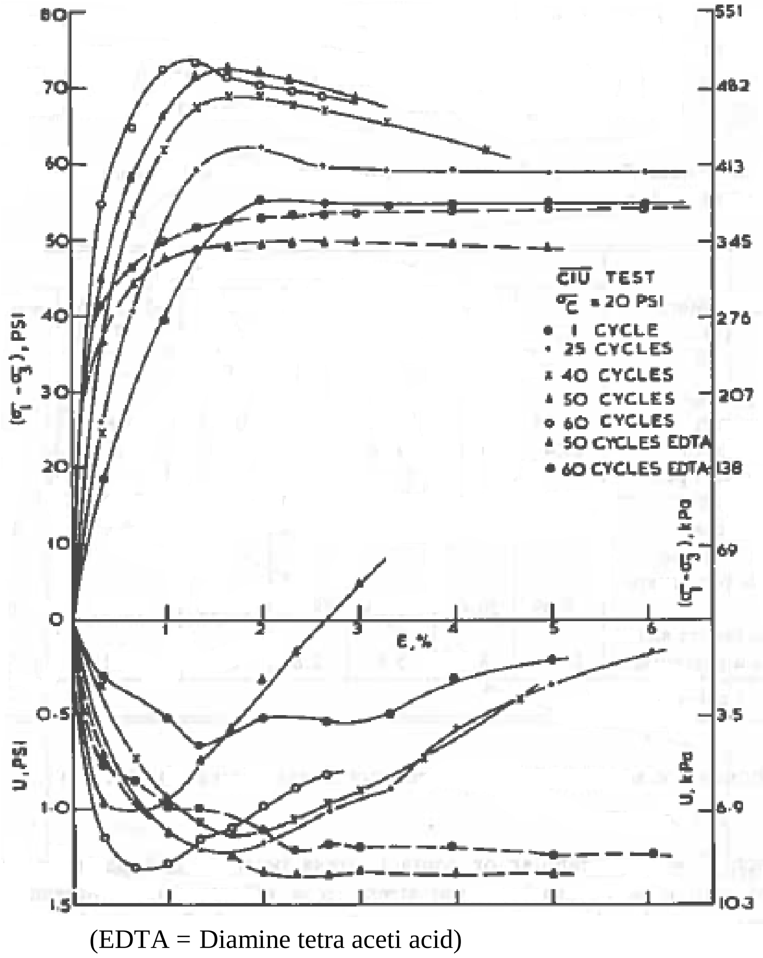


Figure 2.45 Deviator stress versus axial strain curves (b) volumetric strain versus axial strain curves under constant matric suction (50kPa) and constant net confining stress (50kPa) under drying and wetting (Gallage and Uchimura, 2006)

Allam and Sridharan (1981) conducted a series of tests on non-lateritic soil (locally described as red earth, it is composed of about 70% sand, 25% silt and 5% clay, the index properties are: liquid limit: 32.1%, plastic limits = 18.7%, shrinkage limit: 17.5% and specific gravity: 2.6) to investigate the cyclic wetting and drying effect on shear strength of soil, by use of a triaxial test. The specimens' water content was 16% and a dry density was 1.79g/cm^3. The specimens' water content during fully saturated condition was 17.5% which is equal to the shrinkage limit of the soil. The prepared specimens were allowed to first air dry in air-controlled conditions, until there was no change in weight, before being placed in an oven for 24hrs at a temperature 110^{0}C. At the end of first cycle, the specimens were cooled in air and placed on a tray with blotting paper and completely covered with fine sand which was then saturated for 3 days. The process was repeated up to 59 cycles. The results were presented for 1, 25, 40, 50 and 60 cycles of rewetting (Figure 2.46). It was seen that the shear strength progressively increased with number of wetting and drying cycles.



(EDTA = Diamine tetra aceti acid)

Figure 2.46 Deviator stress-strain and pore pressure curves for wetting and drying samples (Allam and Shridharan, 1981)

The swelling and shrinking of expansive soil causes problems to subgrade soil due to seasonal changes in moisture content. Swelling of soil increases volume if water content increases, while volume decreases with the drying out of water; the subgrade will thus experience an undesirable cracking and movement (Dif and Bluemel, 1991). Dif and Bluemel (1991) stated that there are three factors responsible for the fatigue of expansive soils after wetting and drying cycles which are: (i) reorganization of the soil particles that conduct a progressively more intense destruction of internal structure, (ii) loss of lateral confinement due to cracks forming and (iii) the type of clay minerals present in the soil. If a pavement is constructed during wet season or an excessive amount of water is used during compaction of the subgrade, a drying cycle follows the construction as the subgrade reaches moisture equilibrium with the ambient conditions. Alternatively, if a pavement is laid down during the dry season or compacted dry, a wetting process begins following the construction. Both of these processes may result in significant volume change (settlement or swelling). Furthermore, drying and wetting cycles may also result in increased shear deformations in the subgrade under traffic loads (Gupta et al., 2007). The maintenance costs may increase because of reduction of safety factors due to wetting after construction, where the design was based on dry conditions (Uchaipichat, 2010). It has also been reported that the subgrade soil reach to a stabilised point after a certain period of seasonal variation due to repeated wetting and drying cycles (Dif and Bluemel, 1991).

2.7.1 Soil water retention curve

The relationship between soil water content and soil suction is known as the soil water retention curve (WRC) or the soil water characteristic curve (Gardner, 1958; Fredlund and Rahardjo, 1993; Barbour, 1998; Vanapalli et al., 1999b; Pham et al., 2003; Lu and Likos, 2004; Yang et al., 2004; Péron et al., 2007; Fredlund, 2006; Nuth and L.Laloui, 2011). Water retention curve hydraulically and physically signifies how much equilibrium water a soil can hold at the specified suction (Ng and Menzies, 2007). The WRC has two different type curves, one for drying and one for wetting. These different curves demonstrate hysteresis, which can be explained by the complicated nature of soil pore structure. Hysteresis occurs due to size differences between the primary pores and the interconnecting pore throats, changes in the contact angle during drying and wetting and trapped air (Hillel, 1980; Fredlund and Rahardjo, 1993; Tinjum et al., 1997). The curve indicates at any given moisture content how much energy is required to remove a small quantity of water from the soil (Gupta et al., 2007). Figure 2.47 presents a typical

WRC for drying path. When the soil is fully saturated the degree of saturation is $S_r = 1$, if air entry increases than the degree of saturation decreases. The wetting curve showing water adsorption when matric suction reduces and the drying curve characterises water desorption of soil when matric suction increases (Figure 2.48). The characteristics of unsaturated soil can be determined by soil water retention, for example the shear strength of soil, volume change, pore size distribution, coefficient of permeability, and water content all can be calculated from the soil water retention curve (Guan et al., 2010). The WRC follows different paths during drying and wetting where the soil on the drying path has higher water content than the soil on the wetting path at a given matric suction (Fredlund and Rahardjo, 1993).

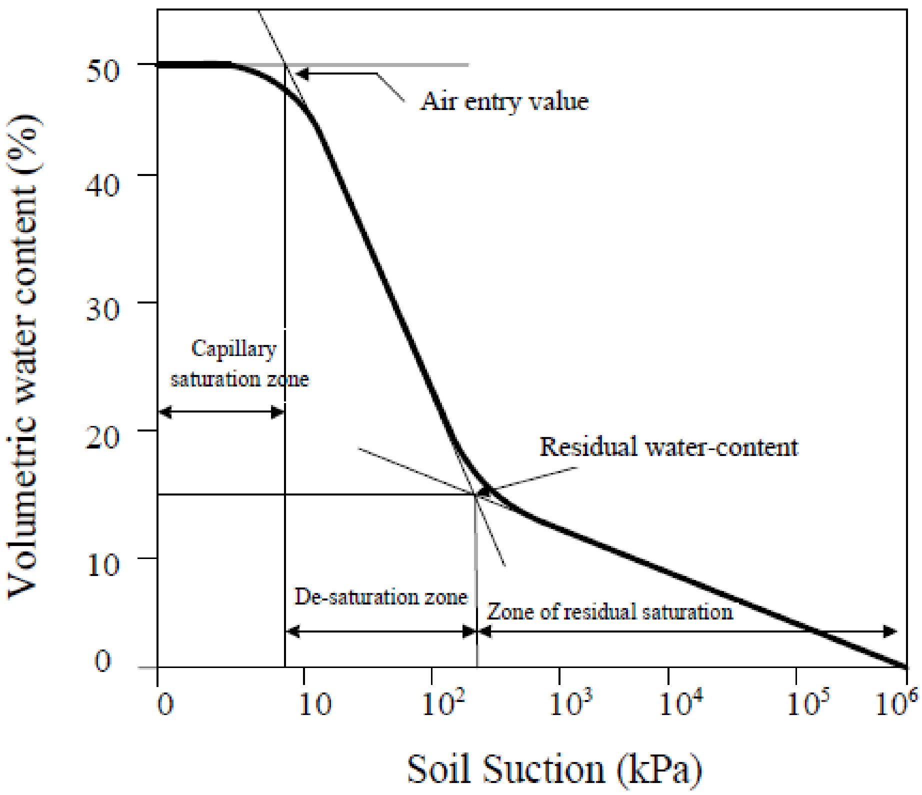


Figure 2.47 WRC showing the regions of desaturation (McQueen and Miller, 1974)

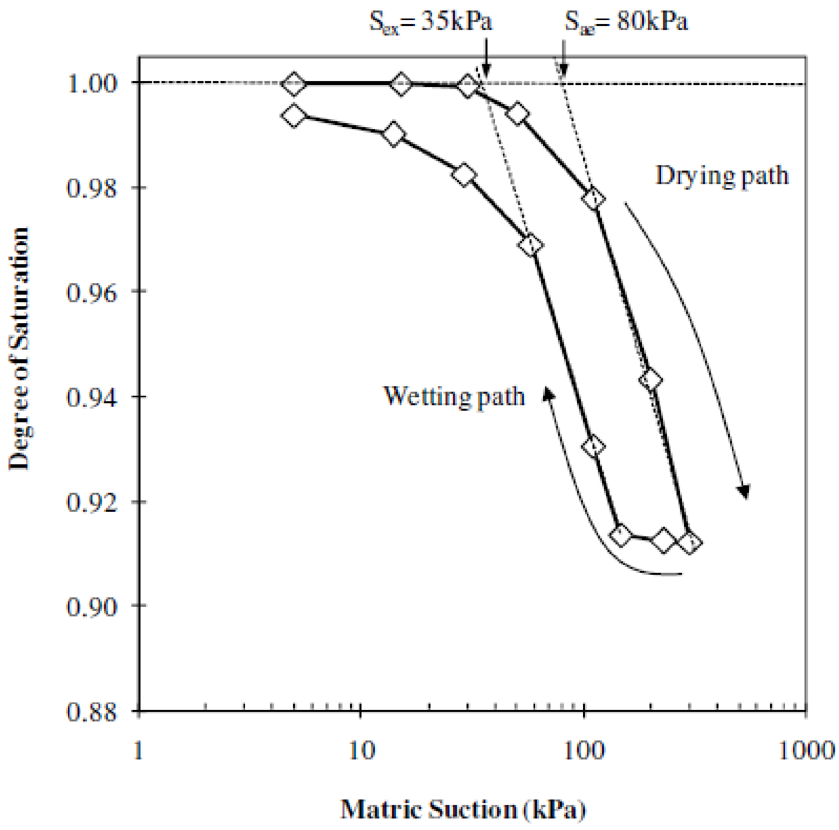


Figure 2.48 Water retention curve of compacted kaolin clay (Uchaipichat, 2010)

With reference to Figure 2.49, the three main identifiable stages of desaturation, which are (i) the boundary effect zone (complete saturation with liquid water), (ii) the transition zone and (iii) the residual state of unsaturation (Vanapalli et al., 1999a). In the boundary effect state, all the soil pores are filled with water. The soil desaturates at the air-entry suction value in the transition zone. In this stage, the soil dries rapidly as the soil suction increases (pendular state) and eventually large increases in suction lead to comparatively small changes in the degree of saturation. In the residual state of saturation can be considered to be the degree of saturation at which the liquid phase becomes discontinuous. Therefore, in the residual stage, the degree of saturation value beyond which it becomes increasingly difficult to remove water from a sample by drainage. However, it is difficult to define the point at which the residual state of saturation is reached (Vanapalli et al., 1999a). Consequently, the shape of the water retention curve may or may not display an hysteresis but is invariably linked to the volumetric information of the soil (Nuth and L.Laloui, 2011).

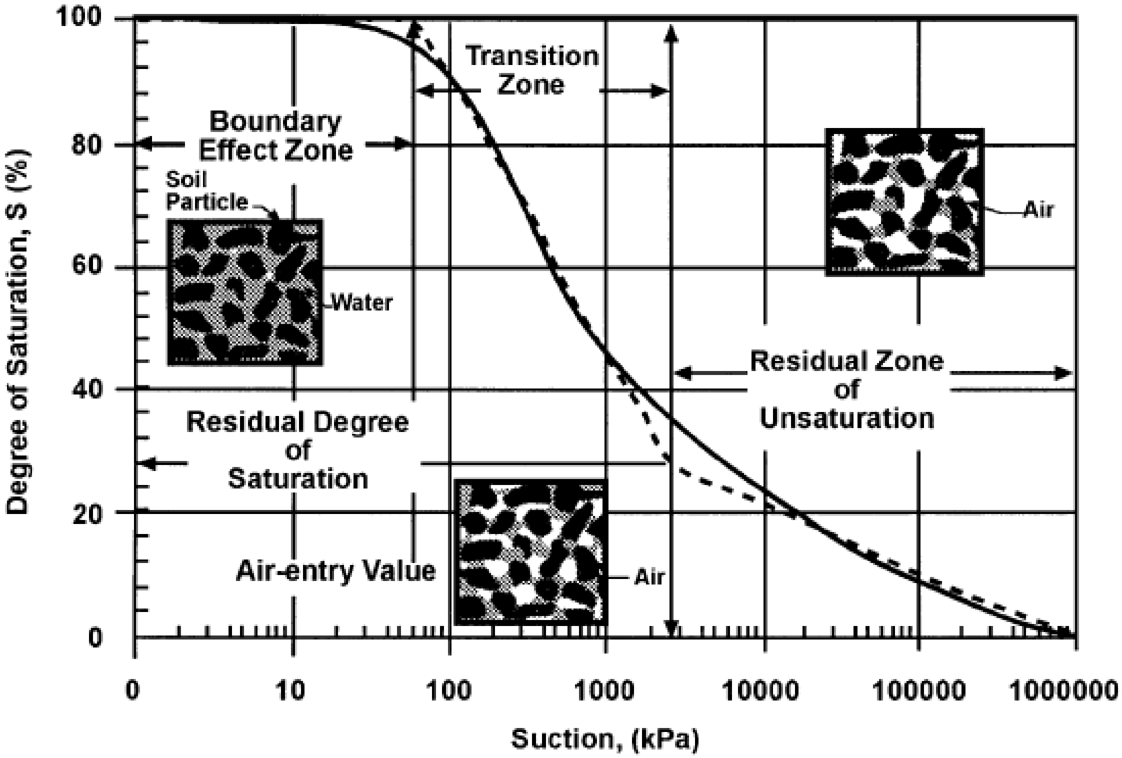


Figure 2.49 Schematic stages of water retention curves presenting zones of desaturation (after Vanapalli et al., 1999)

There are a number of empirical models to describe the WRC of soil. Table 2.5 summarises some of the equations which are reviewed by Leong and Rahardjo (1997).

Table 2.5 A summary of the equations for WRC models (Leong and Rahardjo, 1997)

Authors	Proposed equations for WRC	Parameter
Gardner (1958)	$\theta = \theta_r + \frac{\theta_s - \theta_r}{1 + a\psi^b}$	a, b and θ_r
Brooks & Corey (1964)	$\theta = (\theta_s - \theta_r) + \left(\frac{a}{\psi}\right)^b$	a, b and θ_r
Van Genuchten (1980)	$\theta = \theta_r + \frac{\theta_s - \theta_r}{\left((1 + (a\psi)^b)\right)^c}$	a, b, c and θ_r
Williams et al. (1963)	$\ln\psi = a + b\ln\theta$	a, b
McKee & Bumb (1984)	$\theta = \theta_r + (\theta_s - \theta_r)exp\left(\frac{a-\psi}{b}\right)$	a, b and θ_r
McKee & Bumb (1984)	$\theta = \theta_r + \frac{\theta_s - \theta_r}{1 + exp\left(\frac{\psi - a}{b}\right)}$	a, b and θ_r
Fredlund & Xing (1994)	$\theta = \left[1 - \frac{\ln\left(1 + \frac{\psi}{\psi_r}\right)}{\ln\left(1 + \frac{1{,}000{,}000}{\psi_r}\right)}\right] \frac{\theta_s}{\left\{\ln\left[e + \left(\frac{\psi}{a}\right)^b\right]\right\}^c}$	a, b c and ψ_r
	$\theta = \theta_r + \frac{\theta_s - \theta_r}{\left\{\ln\left[e + \left(\frac{\psi}{a}\right)^b\right]\right\}^c}$	a, b, c and θ_r

θ = normalised water content, θ_s = saturated volumetric water content, θ_r = residual volumetric water content, ψ = suction, ψ_b = air-entry value, ψ_r = suction corresponding to the residual water content and a, b and c = curve fitting parameters.

2.7.2 Modulus-moisture-suction relationship

Resilient modulus is the elastic modulus based on the recoverable strain under repeated loads. Resilient modulus (M_r) is mathematically defined as the ratio of deviator stress to recoverable strain (Figure 2.50).

$$M_r = \frac{\sigma_d}{\varepsilon_r} \tag{2.22}$$

where σ_d is the repeated deviator stress and ε_r is the recoverable strain.

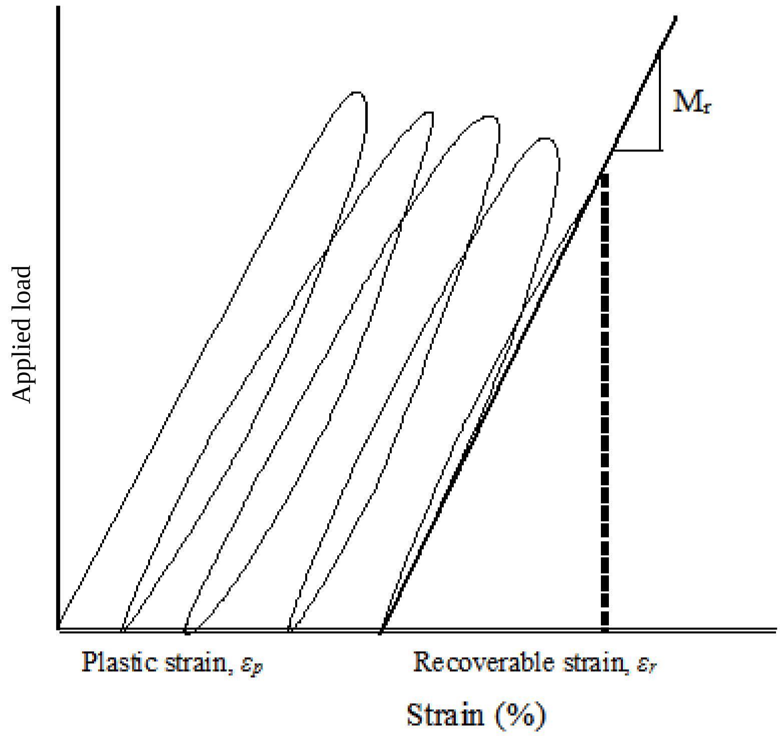


Figure 2.50 explanation of resilient modulus

Resilient modulus is an important parameter to characterise the subgrade soil and for the design and analysis of subgrade. The high matric suction increases the effective stress and therefore decreases the deformation (Yang et al., 2008). The experimental results from Yang et al. (2008) showed that the resilient strain decreases with the steady increases in matric suction (Figure 2.51). However, the resilient modulus does not change with osmotic suction (Khoury et al., 2003). Several authors have demonstrated the strong correlation between the resilient modulus and matric suction (Khoury et al., 2003; Khoury and Zaman, 2004; Sawangsuriya et al., 2008; Yang et al., 2008; Sawangsuriya et al., 2009). The role of soil moisture on track stiffness, particularly track substructure behaviour, is considered to be important (Hunt, 2005). This confirms the fact that matric suction being a fundamental stress state variable, is a much better tool to anticipate the soil behaviour than the soil moisture content (Cary and Zapata, 2011). Although, the changes in modulus of compacted subgrade in response to moisture content and matric suction changes have rarely been studied, particularly after construction (Sawangsuriya et al., 2009). Salem et al. (2003) reported from their experimental study on silty clay subgrade soil that the variation of subgrade modulus and moisture with time followed an inverse function, where the modulus decreased with moisture increase. Zaman and Khoury (2007) investigated the effect of moisture changes and soil suctions on the resilient modulus (eight different soils) that was subjected to a wetting and drying process. Similar results were also found for a given moisture content; the M_r values are higher for a drying cycle than for the wetting cycle. They also observed that the resilient modulus and soil suction qualitatively showed a similar trend due to moisture variations.

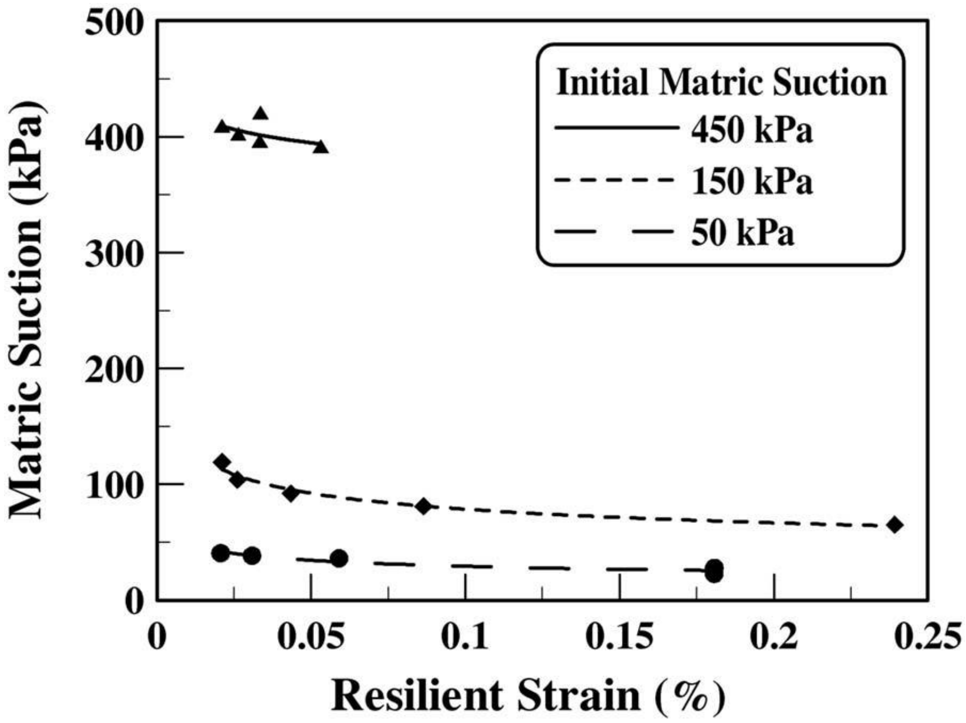
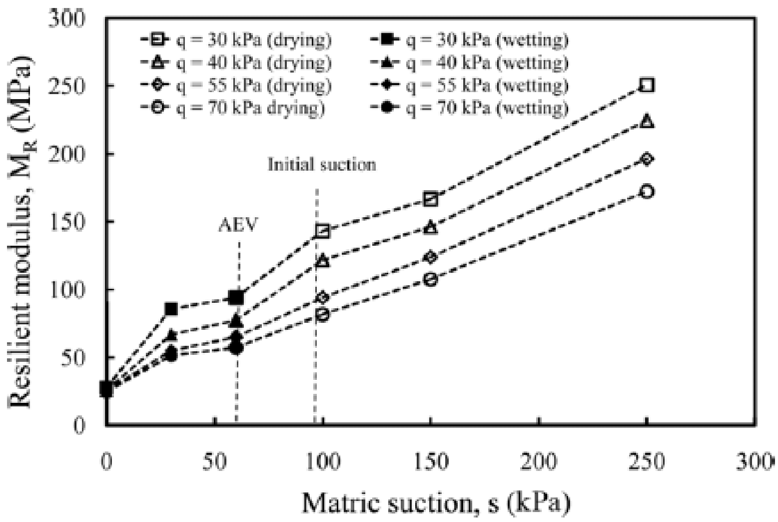
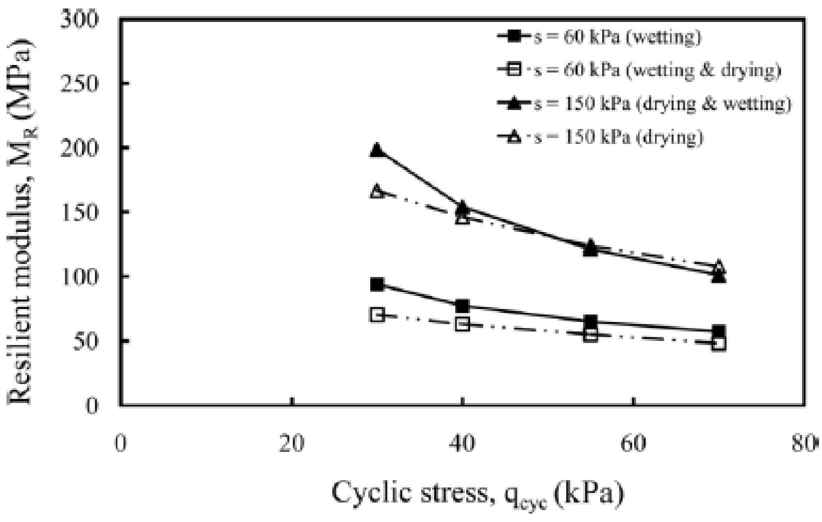


Figure 2.51 Variation of resilient modulus with matric suction (Yang et al., 2008)

Ng et al. (2013) reported that M_r increases significantly with increasing of suction (regardless whether it is following drying or wetting path) is shown in Figure 2.52a. At 30kPa cyclic stress, the M_r increased by approximately 10 times when the suction increased from 0 to 250kPa (at drying path). On the other hand, at the wetting path, the relationship between M_r and suction is nonlinear. At cyclic stress of 30kPa, M_r doubles when suction increased from 0 to 30kPa and yet increased by only 10% increased when suction increased from 30 to 60kPa. However, different results were observed at different suction levels. In the lower suction level (lower than AEV), bulk water dominates the behaviour but when suction increased above AEV then the meniscus water effect dominates the behaviour. Ng et al. (2013) also observed that the M_r is larger in the wetting path compared to the drying path, at the same level of suction (Figure 2.52b). The influence of wetting and drying on soil stiffness is a result of the coupling effect between mechanical and hydraulic behaviour. Unsaturated soil deforms upon suction change and irreversible volume change may occur during cyclic wetting and drying. When suction increases from 150 to 300 kPa and then decreases to 150 kPa, plastic shrinkage occurs and soil density increases. The soil specimen in the wetting path behaves like an overconsolidated soil and results in a greater stiffness (at least in the low cyclic stress range). When cyclic stress increases to 50 kPa at a suction of 150 kPa, M_r measured in the drying path becomes even larger than that measured in the wetting path. The wetting and drying effect not only induces effects of overconsolidation, but also affects the equilibrium soil water content. Therefore, M_r measured in the drying path could be even larger, when the effects of overconsolidation become relatively less important at high cyclic stress levels.



(a)



(b)

Figure 2.52 Influence of (a) suction on resilient modulus and (b) drying and wetting history on resilient modulus (Ng et al., 2013)

Fredlund et al. (1977) proposed, resilient modulus of unsaturated soil is a function of three stress variables, which is

$$M_r = f[(\sigma_3 - u_a), (u_a - u_w), (\sigma_1 - \sigma_3)] \tag{2.23}$$

where u_a is the pore pressure, u_w is the pore water pressure, $(\sigma_3\text{-}u_a)$ is the net confining pressure, $(u_a\text{-}u_w)$ is the matric suction and $(\sigma_1\text{-}\sigma_3)$ is the deviator stress.

Fredlund et al. (1977) also showed two stress state variables that are functions of matric suction (define the linear relationship between the logarithm of the resilient modulus) and the deviator stress (the slope of the line and the intercept). The proposed model is:

$$Log\ M_r = c_{id} - m_{id}(\sigma_1 - \sigma_3) \tag{2.24}$$

where c_{id} and m_{id} are functions of matric suction and σ_1 and σ_3 are the major and minor principle stresses, respectively.

Yang et al. (2005) proposed a model which is a combination of externally applied stress and matric suction, on the prediction of the resilient modulus, by using an effective stress concept which is similar to that used for saturated soils. The model based on Bishop (1959) effective stress parameter represents the moisture conditions of the soil.

$$M_r = k_1(\sigma_d + \chi_m \psi_m)^{k_2} \tag{2.25}$$

where χ_m is the Bishop's parameter and k_1 and k_2 are regression parameters.

Liang et al. (2008) proposed a similar model based on the concept of effective stress.

$$M_r = k_1 p_a \left[\frac{(\theta + \chi_m \psi_m)}{p_a}\right]^{k_2} \left[\frac{\tau_{oct}}{p_a} + 1\right]^{k_3} \tag{2.26}$$

where θ is the bulk stress = $\sigma_1+\sigma_2+\sigma_3$, where σ_1, σ_2 and σ_3 are the three principal stresses; τ_{oct} is octahedral shear stress = $\sqrt{2}/3(\sigma_1-\sigma_3)$ (for triaxial condition), ψ_m is the matric suction, χ_w is the Bishop's parameter, p_a is the atmospheric pressure and k_1, k_2, k_3 are regression constants.

Cary and Zapata (2011) developed a model which is a function of three stress state variables. The variables are bulk stress (θ), octahedral shear stress (τ_{oct}) and matric suction (ψ_m).

$$M_r = k_1' p_a \left(\frac{\theta_{net} - 3\Delta u_{w-sat}}{p_a}\right)^{k_2'} \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3'} \left(\frac{(\psi_{m_0} - \Delta\psi_m)}{p_a} + 1\right)^{k_4'} \quad (2.27)$$

where p_a is the atmospheric pressure, $\theta_{net} = \theta - 3u_a$ is the net bulk stress, where u_a is the pore air pressure, $\Delta u_{w\text{-}sat}$ is the build-up pore pressure under a saturated condition. τ_{oct} is octahedral shear stress = $\sqrt{2}/3(\sigma_1\text{-}\sigma_3)$, ψ_{m0} is the initial matric suction, $\Delta\psi_m$ relative change in soil matric suction with respect to (ψ_{m0}) due to pore-water pressure build-up under unsaturated condition ($\Delta u_{w\text{-}sat}$ = 0) and $k'_1 \geq 0$, $k'_2 \geq 0$, $k'_3 \geq 0$ and $k'_4 \geq 0$ are regression constants.

2.7.3 Volume change behaviour

Soil suction has great influence on the volume change behaviour of unsaturated soils. Volume change calculations in compacted soil are important for the analysis, design, and performance of geotechnical work. The volumetric behaviour depends on soil type, structure, initial density, total stress, pore water pressure and the water content. In compacted soil, volumetric stability is not predictable due to suction changes and the wetted compacted soil can experience volume decrease (or collapse), volume increase (or swelling) or can remain unchanged (Cerato et al., 2009; González and Colmenares, 2006). In order to explain the volume change behaviour of unsaturated soils, Bishop and Blight (1963) used two state variables which are (i) net stress (σ - u_a) and (ii) matric suction (u_a - u_w). In fully saturated soil, total volume changes are equal to the changes of water volume as both solid and water phases are regarded as incompressible; volume changes are mainly dependent on the in or out flow of water. On the other hand, in unsaturated soil water volume change is not equivalent to total volume due to the presence of air in the soil; therefore, soil suction and stress both need to be considered in order to understand the unsaturated soil behaviour. The mechanical behaviour of compacted soil is highly dependent on the volume-mass soil properties such as void ratio e, gravimetric water content w and degree of saturation S_r. The theory of unsaturated soil mechanics provides a relationship between change of volume mass properties and changes in the stress state of a compacted soil (Fredlund and Morgenstern, 1977; Pereira and Fredlund, 2000). Unsaturated soil may either swell or collapse upon wetting if the confining stress is necessarily low (swelling) or high (collapse) (Alonso et al., 1990). Therefore, swelling and collapse behaviour depends upon when soil imbibes water and the magnitude of the mean net stress.

Collapse is a significant volume reduction upon wetting of unsaturated soil under load (Matyas and Radhakrishna, 1968; Delage et al., 2005; Nelson et al., 2011) (Figure 2.53). The collapse behaviour occurs due to the decrease of suction during inundation and under load (Matyas and Radhakrishna, 1968; Tadepalli and Fredlund, 1991; Houston et al., 2002). According to Barden et al. (1973), there are three conditions required to be observed in soil collapse: (i) an open potentially unstable unsaturated structure, (ii) an high enough value of applied stress component to develop a metastable condition and (iii) a higher value of suction (or other bonding or cementing agent) to stabilise intergranular contact and which are reduced on wetting, thereby leading to collapse.

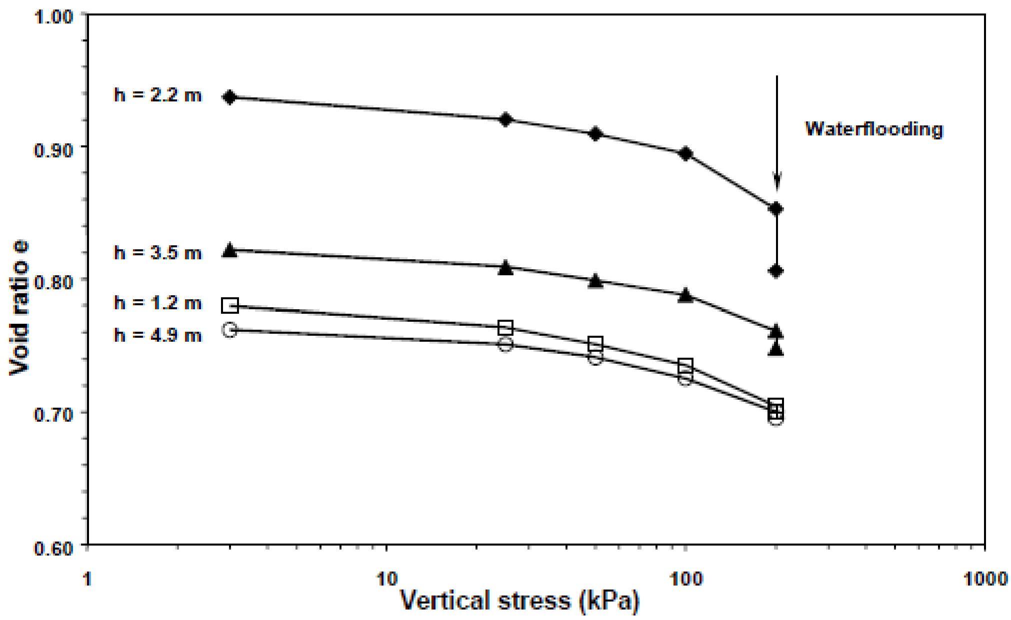


Figure 2.53 Collapse test by simple oedometer technique (Delage et al., 2005)

It is generally considered that only sandy or silty soils exhibit collapse but it has been reported that compacted soils in general show collapse (Barden et al., 1973). It has been reported that the dry density of unsaturated soil lower than 1.6 Mg/m^3 is likely to collapse (Meckechnie (1989) in Tadepalli and Fredlund, 1991). The collapsible soil is found in manmade or natural earth slopes, underlying a foundation or subgrade soil. Collapsible soil usually shows high stiffness and shear strength in a naturally dry state, whereas the soil experiences a large volume reduction upon wetting resulting from a soil suction decrease and weaker bonding between solid particles (Houston et al., 2002). Collapse occurs in a relatively short period of time in response to infiltration of water at a constant vertical stress; resulting in a drastic rearrangement of the soil particles as a result of a significant reduction in total volume of soil mass (Tadepalli and Fredlund, 1991).

Zakaria (1994) explained swelling or collapse behaviour during wetting by considering the fabric of unsaturated compacted clay soils. The increase of pore pressure during wetting leads to a decrease of the local value of effective stress within the saturated microstructure of clay packets; which is why wetting always generates swelling of the individual particles. At lower stress collapse does not happen because the soil is able to support even though softening of the inter-packet contacts. At higher stress levels collapse happens as the microstructure is unable to give support after softening of the inter-packet contacts.

Cui and Delage (1996) carried out a series of suction controlled triaxial experiments on compacted Aeolian silt. The investigation shows that the plastic compressibility of the soil decreased with increases of suction. Rampino et al. (1999) also found similar behaviour from a series of suction controlled triaxial tests on compacted silty sand. Pereira and Fredlund (2000) described the behaviour of volume change of collapsing soil in three distinct phases during the wetting process is shown in Figure 2.54. In the first phase, the deformation happens at high matric suction and it is characterised by small volumetric deformations of the soil in response to a reasonably large reduction in matric suction. This behaviour indicates small values for the soil compressibility as the matric suction changes. This phase is called the precollapse phase. In the second phase, termed the collapse phase, deformation occurs at intermediate values of matric suction; the collapse is characterised by significant volumetric deformation in response to decreasing of matric suction. This behaviour can be explained in terms of a combination of additional rearrangements and by the incidence of local shearing of both the connecting bonds and clay aggregations as the matric suction decreased. The third phase of deformation follows at low matric suction. This phase, is termed the post-collapse phase, indicates an insignificant soil compressibility in response to further reduction of matric suction,

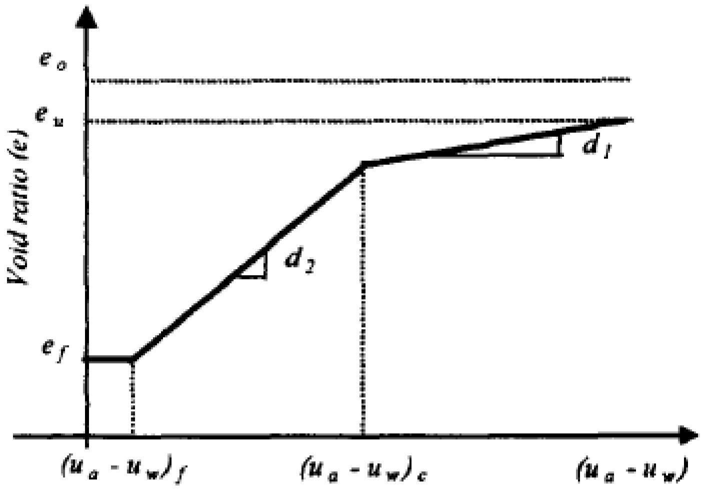


Figure 2.54 volume change behaviour of collapsing soil during wetting (Pereira and Fredlund, 2000)

Collapsible behaviour of compacted and cohesive soils depends on the percentage of fines, initial dry density, initial water content, energy, imposed stress and compaction process (Ng and Menzies, 2007). The compacted collapsible or metastable soil structure is supported by the micro-forces of shear strength; the bonds are highly dependent upon capillary action; the bond strength becomes weak with increasing water content and at a critical degree of saturation, structure collapse will occur (Murthy, 2002).

Often collapsible soils go undetected until a structure or roadway is already in place. Even if collapsible soils are identified prior to construction, lack of knowledge of potential sources of wetting can lead to incomplete mitigation of the problem. For example, a rising groundwater table may not be given due consideration, or infiltration may extend to a greater depth than assumed. Pavement sections are particularly vulnerable to damage, because the large area covered makes site exploration and preconstruction mitigation more difficult and costly (Houston et al., 2002). The dry density and water content of soil specimens at the time of compaction are generally considered as the primary soil properties that control the amount of collapse. At the same time, an initially unsaturated condition is a prerequisite for collapse (Houston et al., 2002). Collapse mechanisms differ considerably from the classical consolidation process. In the consolidation process, the total volume change of saturated soils occurs as a transient process. Collapse, on the other hand, appears to occur in a relatively short period of time in response to the infiltration of water at a constant vertical stress. Collapse can cause a radical rearrangement of the soil particles, resulting in a significant reduction in total volume of the soil mass. The collapse phenomenon is primarily related to the reduction of the matric suction during inundation. González and Colmenares (2006) noted from their experiments that the collapse of the sample occurred during the final stage due to water entry at low suction which filled the inter aggregates spaces and rearrange the particles, thereby causing a high possibility of collapse.

2.8 Influence of moisture content and suction variations on subgrade behaviour

It has been recognised that the suction is a fundamental variable in the mechanical behaviour of unsaturated soil. Suction is generally influenced by climatic conditions rather than loading conditions (Rahardjo and Leong, 2006). It has been reported that the soil suction changes significantly with changes of moisture content (Heydinger and Randolph, 1998). Zhan and Ng (2006) observed that the suction effect on the shear strength of expansive soil is found in two ways, one is capillary force to interparticle normal stress and the second is through soil dilatancy.

Suction increases the stiffness of unsaturated compacted soil as the matric suction generates an additional effective confining pressure with the soil structure (Mendoza and Colmenares, 2006). Matric suction is considered one of the variables that controls the shear strength of soil (Farouk et al., 2004). There is evidence that the soil suction affects soil properties, such as void ratio and the degree of saturation. Sudjianto et al.(2011) noted that the suction has great influence on both vertical and horizontal swelling soil behaviour and also volumetric swelling behaviour is shown in Figure 2.55. Therefore, an increase of suction value reduces vertical, horizontal and volumetric swelling movements.

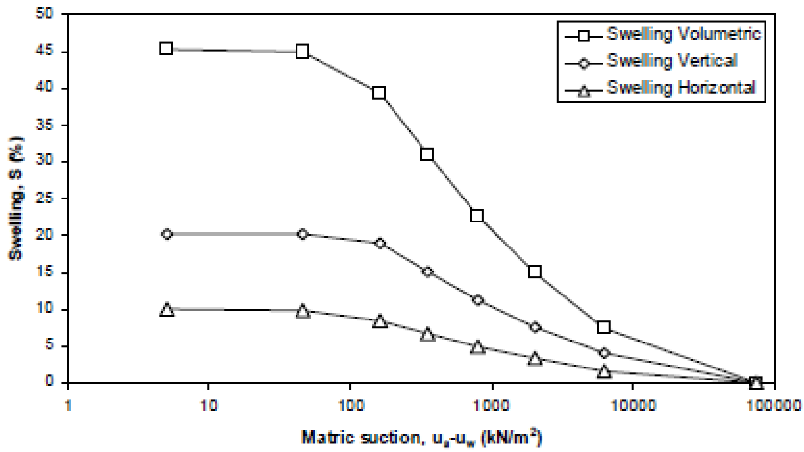


Figure 2.55 Relationship between swelling with metrics suction clay soil (Sudjianto et al., 2011)

The moisture content of subgrade soil is susceptible to a fluctuation in the water table, water infiltration and evaporation and the variation of moisture's effect in subgrade effect on subgrade performance (Sawangsuriya et al., 2009). During periods of rain, water content increases which can cause subgrade distress or even shear failure under cyclic loading (Liu and Xiao, 2010) (Figure 2.56). Plastic deformation depends on seasonal variation and weather, load level and subgrade location (Li and Selig, 1996). Oloo and Fredlund (1995) stated that the seasonal variations in moisture content in expansive soil subgrade caused damage from changes in bearing capacity and movements of subgrade soil. The long-term performance of the subgrade depends on the soil properties, which varies extensively due to climatic effect (Heydinger and Randolph, 1998). There is evidence that the subgrade resilient modulus increases with decreases in moisture content and increase in suction (Jin et al., 1994; Heydinger and Randolph, 1998; Parreira and Goncalves, 2000). It is widely accepted that the water content of the subgrade has an important role to play in the mechanical behaviour of the track's substructures. If water is entrapped in the sub-structure, pore pressure can increase significantly under cyclic loading; consequently the shear strength and stiffness of sub-structures can decrease (Selig and Waters, 1994; Alobaidi and Hoare, 1996; Huang et al., 2009; Trinh et al., 2012). Moisture plays a significant role in the modulus of subgrades. An increase in the water content of subgrade layers decreases the modulus values as a result, so shortening the service life of subgrade and causing a significant increase in maintenance costs (Ksaibati et al., 2000). The soil moisture changes with the temperature; it influences particle bonding and quantity of water content. In freezing temperatures, the subgrade modulus can increase as a soft subgrade can be converted to a rigid material, whereas during a thawing period, the soil becomes soft and the shear strength decreases significantly (Salem et al., 2003). The changing of negative pore water pressure causes a change in both the shear strength and volumetric behaviour of the soil, which is why near the ground surface soils are called 'problematic' soils. These include collapsible soils and expansive or swelling soils (Fredlund, 2006). Collapsible or metastable soils cover approximately 16% of the earth's land surface (Reznik, 2000).

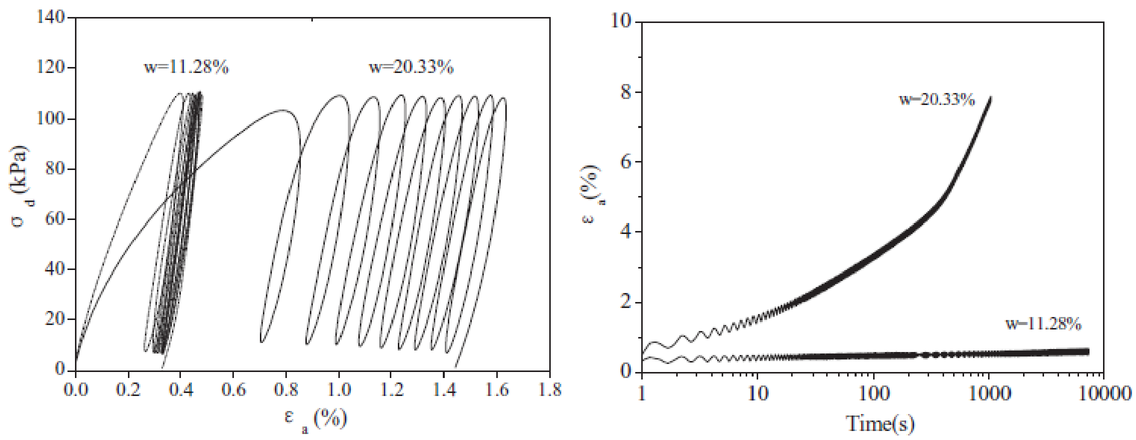


Figure 2.56 Comparisons of deformations at different water contents (a) stress-strain curve and (b) strain history (Liu and Xiao, 2010)

2.9 Summary

A review of literature relevant to the current research has been presented in this chapter. The literature review has focused on railway subgrade problems resulting from uncontrolled overflow and hydro-mechanical behaviour of unsaturated soil including soil water retention characteristics, stress state variables and volumetric behaviour. The laboratory experimenting of unsaturated is also reviewed and presented.

Compacted subgrade soils are considered as unsaturated soil but the assessment is not studied based on unsaturated soil mechanics. The literature has highlighted the importance of soil suction. Soil suction plays an important role in unsaturated soil, for example higher suction signifies higher strength and stiffness, whereas lower suction indicated lower strength and stiffness. Cyclic wetting and drying is another factor not yet considered in the determination of hydro-mechanical behaviour of subgrade soil.

Railway track in the UK often experiences flooding, however, there is little documentation regarding the impact of flooding, particularly post flooding track behaviour. It is impossible to prevent water entering inside the track but it is possible to direct the water away from the track quickly.

The growing demand for high-speed trains will require the planning of strong support from the substructure and appropriate maintenance. Therefore, it is important to understand and predict subgrade behaviour in different conditions and circumstances.

CHAPTER THREE- MATERIALS AND EXPERIMENTALTECHNIQUES

3.1 General overview

This chapter describes the materials, experimental techniques and procedures used in this research project. First, the basic characteristics of ballast are described. Particle size distribution and a large shear box (300mm×300mm) test were conducted prior to the main test to ensure they are in standard size and strength. However, in the research main, focussed on subgrade soil behaviour. The experiment is followed by subgrade soil experimental techniques and procedures of using a range of tests including the plate load test, pressure-plate test, filter paper test and oedometer test. The chapter details the main investigation which was performed in the Geopavement & Railway Accelerated Fatigue Testing (GRAFT) facility, thereby enabling investigation of different track conditions at full-scale. In this research, the changes of the soil behaviour in different conditions, due to flooding, were investigated. A summary of the experiments undertaken, together with their purpose, is presented in Table 3.1. Figure 3.1 presents a schematic diagram of the experimental programme.

Table 3.1 Summary of the performed tests and their purposes

Test	Purpose of test
Particle size distribution	To determine ballast size
Large shear box	To obtain the shear strength of the ballast in both dry and wet conditions
Plate load test	To determine the stiffness of the subgrade
Filter paper	To determine matric suction, total suction and obtained water retention curves
Pressure plate	To measure the water retention curves
Double and single oedometer test	To investigate the collapse behaviour of subgrade soil
GRAFT test	To investigate track performance in unsaturated and saturated conditions (before and after flooding)

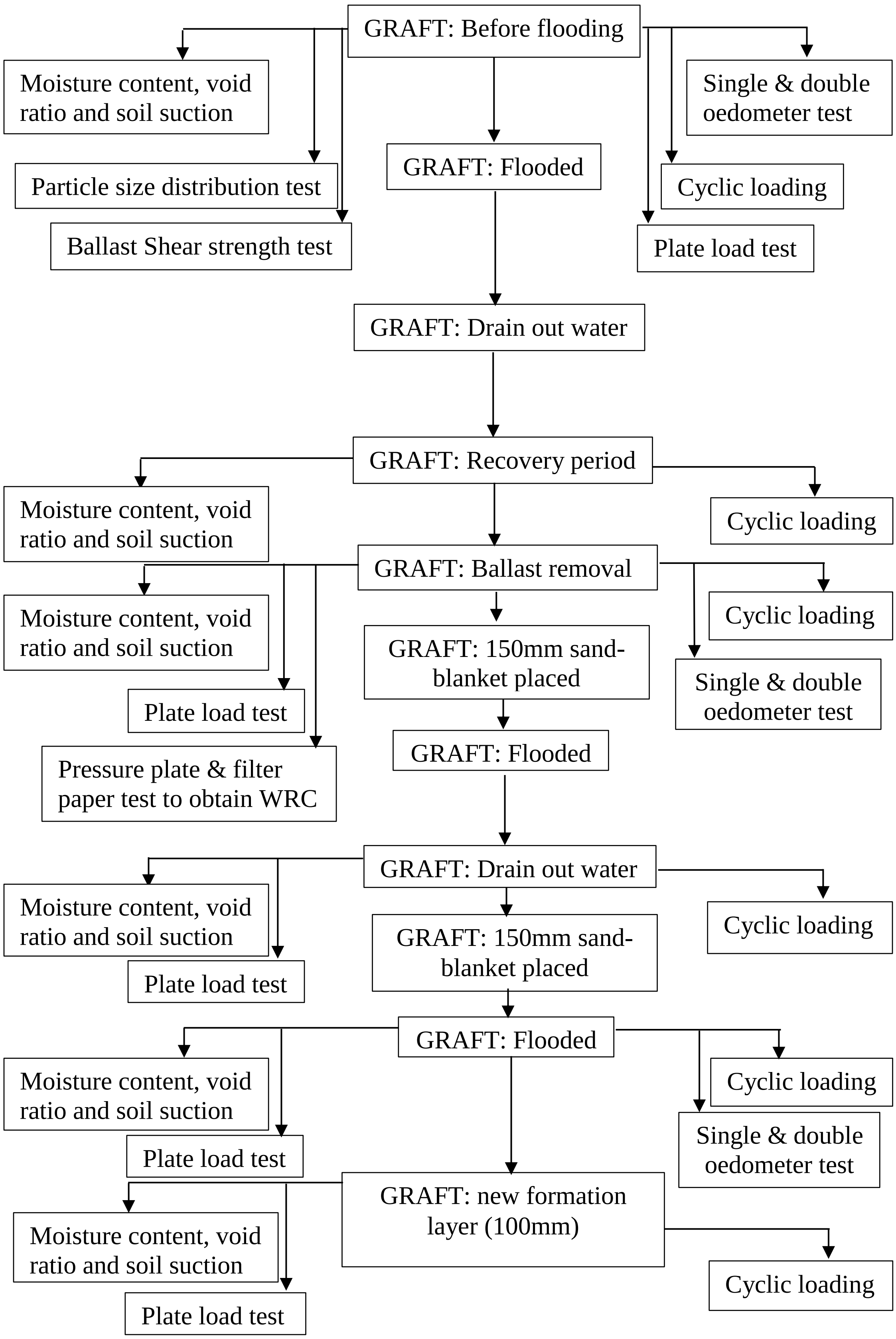


Figure 3.1 Schematic diagram of experimental programme

3.2 Ballast characterisation

The ballast material was supplied from Cloburn Quarry Company Ltd. The ballast was tested before undertaking the main test in GRAFT to make sure it was the right size, the right strength and clean. This was particularly the case after flooding as some of the ballast was not suitable for reuse, in this instance the ballast was cleaned by jet-washing. It is well documented that foul ballast can cause ballast degradation, reduce shear strength, induce settlement, decrease of hydraulic conductivity and reduce overall track performance (Selig and Waters, 1994; Suiker et al., 2005; Indraratna et al., 2006; Aursudkij et al., 2009; Huang et al., 2009; Indraratna et al., 2013b; Indraratna et al., 2014).

3.2.1 Particle size distribution test

The particle size distribution (PSD) test was performed to ensure the ballast has a consistent mixture of sizes based on the specification provided by Network Rail line specification RT/CE/S/006 (2000). The test was undertaken according to British Standard BS812-103.1 (1990a) as specified in the Network Rail specification cited above.

For BS 812 (1990) specifications, 35Kg is the minimum mass of ballast required to do the test. Therefore, 7 individual 5kg ballast sample were taken for sieving. The samples were placed on nested set of sieves which were vibrated for 10 minutes by a vibrating shaker. Figure 3.2 shows the obtained PSD curve together with Network rail standard for maximum and minimum specifications for the ballast. Generally, the track ballast is required for a consistent mixture for the PSD test, with ballast from 28mm-50mm according the sieve test specified in BS812 Section 103.1 (1990). The PSD curve was within the Network Rail grading limit hence the ballast complied with standard specifications. The Coefficient of Uniformity (Cu) was 1.42 which is less than 2 signifying that the ballast was uniformly graded and the coefficient of curvature (Cc) of approximately 1, which indicates most of the ballast was between D60 and D10 sizes.

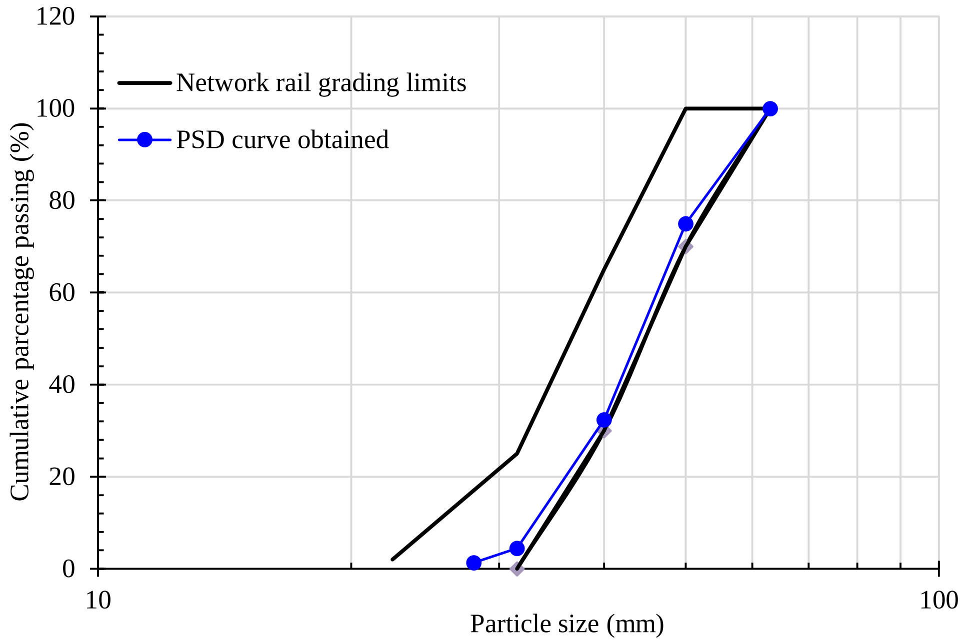
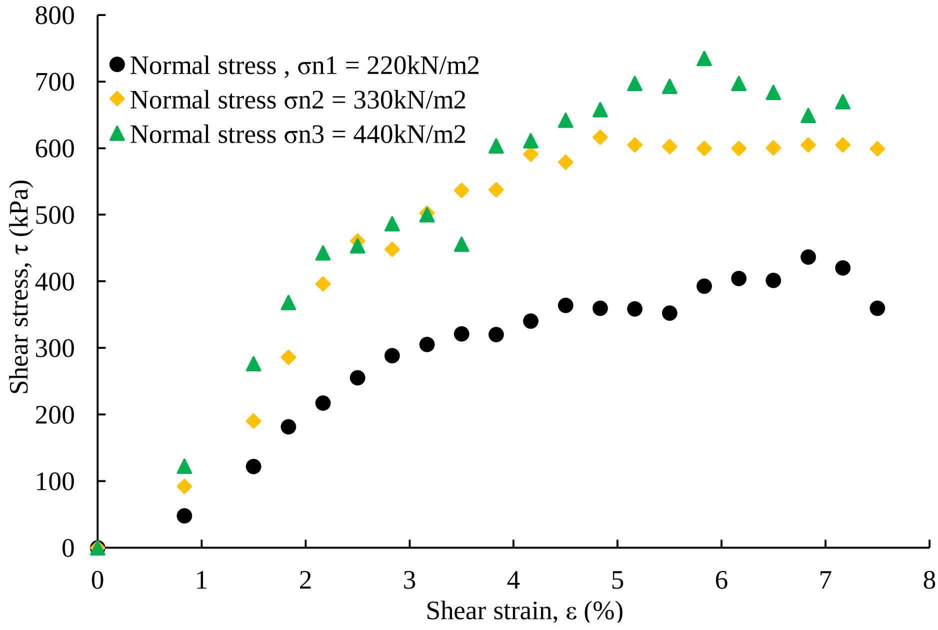


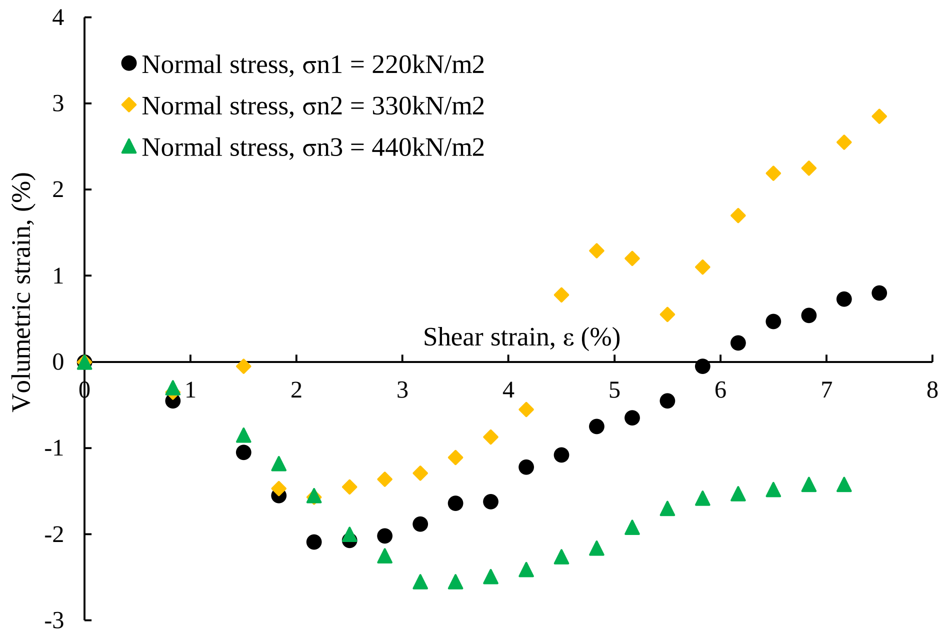
Figure 3.2 Network rail specified and obtained PSD curve

3.2.2 Large shear box test

A shear box (300mm×300mm) test was conducted to determine the shear strength of ballast. The test followed BS 1377: Part 7:1990. To achieve a ballast bulk density of 13.6kN/m^3, the ballast was compacted in three layers. Vertical loads of 220kN/m^2, 330kN/m^2 and 440kN/m^2 were applied with a strain rate 2.5mm/min at a constant rate until shear failure of the sample. Figures 3.3 and 3.4 present shear strength behaviour for dry and wet ballast respectively. The peak angle of shearing resistance was determined as 56^o for the dry ballast and 54^o for the wet ballast, is shown in Figure 3.5. Generally, peak internal friction angles new ballast range from 48^o to 55^o but are reduced to approximately 44^o after train loading on the track because of the problem of particle breakage (Indraratna et al., 1998; Suiker et al., 2005; Indraratna et al., 2006; Huang et al., 2009). The test was conducted on wet ballast but the results did not show any significant difference. McDowell et al. (2005) also did not find any significant differences in the ballast test; in either dry or wet box tests. There are some limitations associated with a shear box test, such as (a) the drainage system cannot be controlled, and, (b) the failure may not occur along the weakest plane due to the predetermined horizontal failure plane.

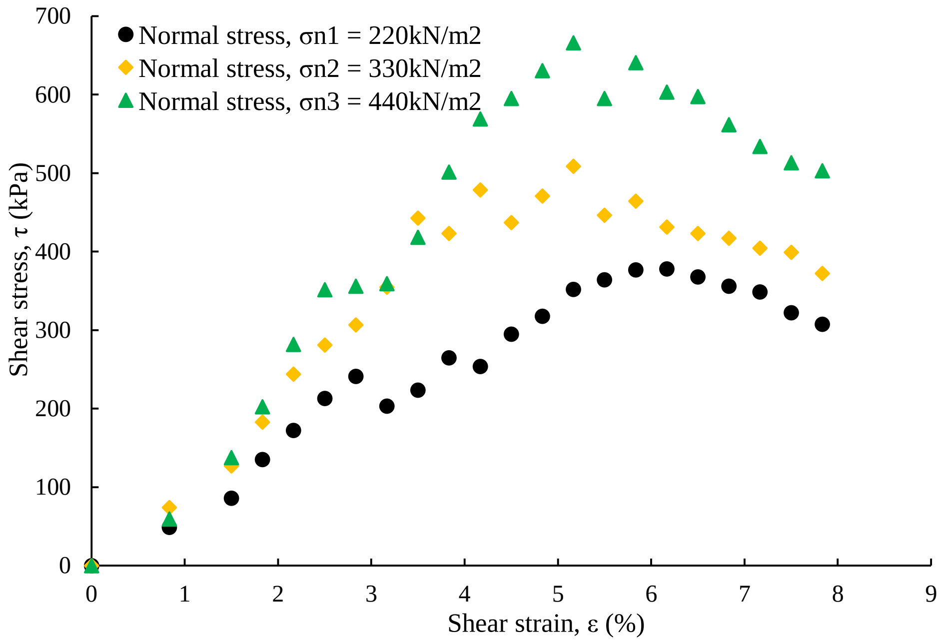


(a)

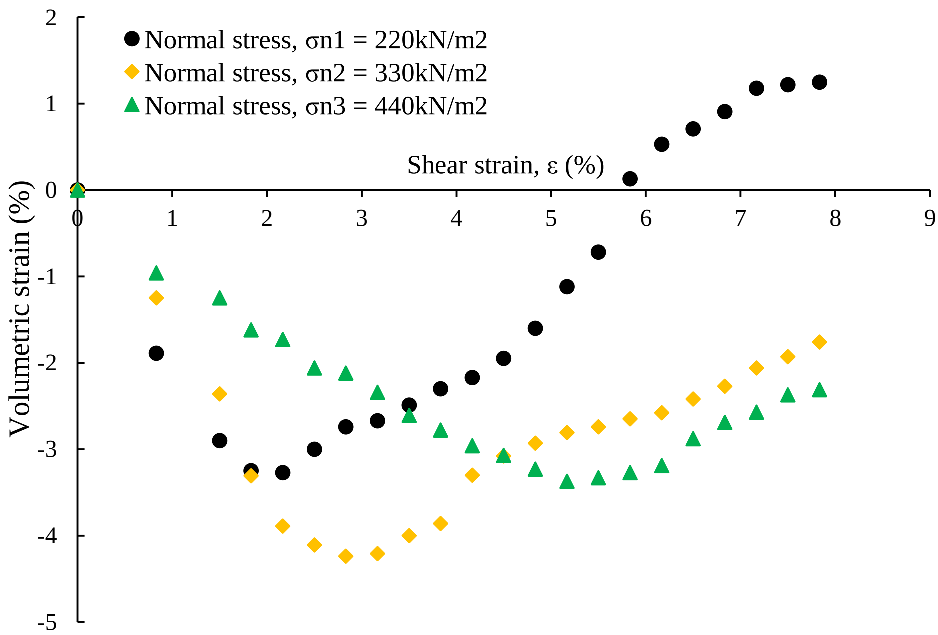


(b)

Figure 3.3 (a) Shear stress-strain curves; (b) Volumetric strain versus shear strain curves for the dry ballast



(a)



(b)

Figure 3.4 (a) Shear stress-strain curves; (b) Volumetric strain versus shear strain curves for the saturated ballast

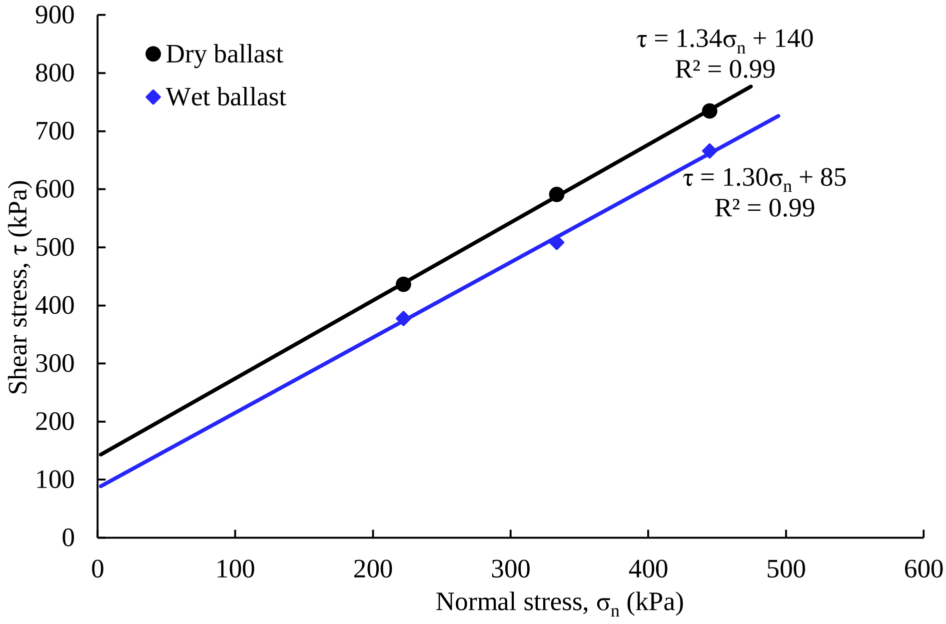


Figure 3.5 Comparison shear strength envelop between dry and wet ballast

3.3 Subgrade soil characterisation

Kaolin clay was used as a subgrade soil, as it is suitable for the investigation in both saturated and unsaturated conditions. On the other hand, it has a low permeability; therefore, the entire subgrade soil does not need to be replaced after each flooding test. The characteristics of the soil are given in Table 3.2:

Table 3.2 Subgrade characteristics (Kennedy, 2010)

Characteristics	Value
Specific Gravity	2.64
Maximum dry density (Mg/m^3)	1.54
Optimum moisture content (%)	23.8
Liquid limit (%)	55.0
Plastic limit (%)	32.0
Plasticity index (%)	23.0

3.3.1 Plate load test

The plate load test (PLT) was conducted to evaluate the subgrade stiffness, which was undertaken in accordance with BS EN 1997-2: 2007. A typical plate load test result is shown in Figure 3.6. A series of stacked circular plates was placed in the middle of the tank and the corresponding vertical deflection of the bottom plate was measured (Kennedy, 2010). The diameter of the steel circular plate on the subgrade surface was 440mm; it was overlaid by a 400mm diameter load cell and three 300mm diameter steel plates, as shown in Figure 3.7. Two linear variable displacement transducers (LVDT's) were placed on the bottom plate to measure the deflection. The influence depth PLT is considered to be almost two times the diameter of the plate (Ping et al., 2002); therefore the test covered the entire depth of subgrade in the GRAFT.

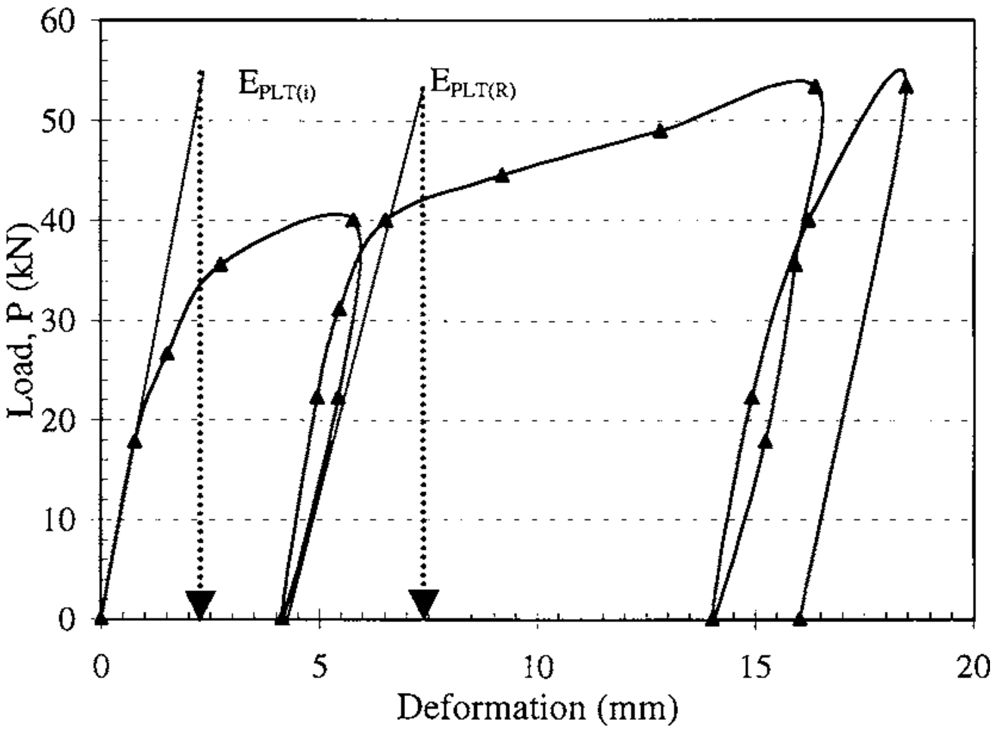


Figure 3.6 Typical plate load test result (Alshibli et al., 2005)

At the beginning of the test, five monotonic load cycles were applied at a rate of 1kN/s and followed by 50 cycles applied at a rate of 0.1Hz to obtain the load-deflection curve. The data were recorded at 30Hz. The applied load for the test was 15kN. This value was calculated to avoid any substantial plastic settlement of the subgrade surface as well as maintaining a stress level of approximately 100kPa underneath the bottom plate. Generally, the subgrade experienced stress of 100kPa under the sleeper (Brough et al., 2003a; Okada and Ghataora, 2002). The results of this series of test are presented in Chapter 6.

Figure 3.7 A typical plate load test in the GRAFT

The subgrade modulus, E_{PLT}, can be calculated from the following equation (3.1) below (Alshibli et al., 2005; Kennedy, 2010). This equation was used to measure both the initial tangent modulus and reloading tangent modulus from the reloading curve. In both cases, the second cycle was used to avoid any problems associated with initial set-up effects.

$$E_{PLT} = \frac{2P\,(1-v^2)}{\pi r \delta} \tag{3.1}$$

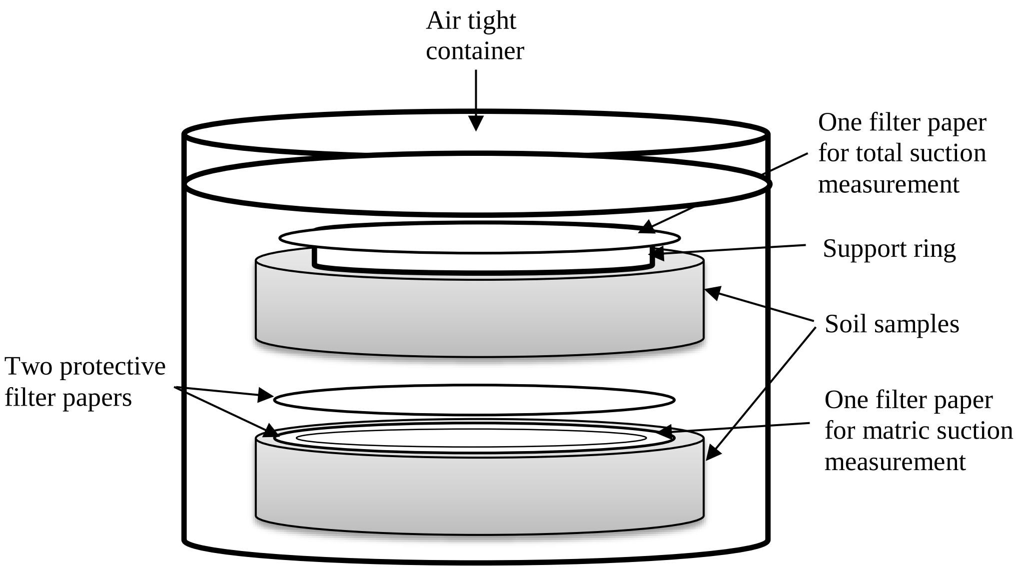
where E_{PLT} is Young's elastic modulus, P is applied load, r is plate radius, v is Poisson's ratio and δ is deflection of plate. In this experiment, the Poisson's ratio was considered to be 0.30 and 0.49 for unsaturated and saturated clay subgrade, within GRAFT (Bowles, 1997).

The plate load test was performed to measure the stiffness of the subgrade before and after flooding. The soil suction was measured by the filter paper to determine the relation between suction and stiffness, as described in Chapter 6.

3.3.2 Filter paper test

Soil suction measurement is an important factor in unsaturated soil behaviour. Seasonal moisture movement changes the soil suction, and this change in soil suction provides important information for many engineering applications (Bulut et al., 2001). The filter paper method is an inexpensive, simple and reasonably accurate method to measure soil suction. It was originally used by soil scientists and agronomists as a suction sensor and dates back to the work of Schull in 1916 (Marinho, 1994) and is now widely accepted and employed in the geotechnical engineering field for suction measurement (Chandler et al., 1992; Ridley and Burland, 1993; Houston et al., 1994; Leong et al., 2002; Bulut et al., 2001; Marinho and Oliveira, 2005; Rahardjo and Leong, 2006).

The filter paper method involves placing a piece of filter paper between two larger sized protective filter papers alongside a soil sample and with another one filter paper on top of the sample. If the filter paper is contact in the middle of the two soil specimens that through menisci of water formations gives the matric suction. In order to measure the total suction, a piece of filter paper was placed on top of the sample without direct contact. The soil specimen is placed in an airtight container at relatively constant temperature which was 25^{0}C to achieve moisture equilibrium condition between the filter paper and soil specimen. Generally, the filter paper comes into equilibrium with the soil either through vapour (total suction) or fluid flow (matric suction); at the state of equilibrium state the soil suction value and the filter paper suction value is the same. Figure 3.8 presents a schematic diagram of the filter paper test.



(a)

Figure 3.8 (a) A schematic diagram and (b) a prepared sample for the filter paper test

In this research, Whatman's No 42 filter paper was used. Leong et al. (2002) reported that the Whatman's 42 is comparatively better than Schleicher and Schuell No 589 as Whatman's 42 is more consistent (Leong et al., 2002). The soil samples were collected from the GRAFT to measure the total and matric suction. The procedure in this experiment was undertaken based on ASTM D 5298 (1997) and Bulut et al. (2001). After the equilibrium period, the filter paper was quickly removed from the sample. The water content of filter paper can change due to equalising with the water content of surrounding air, the whole process should be accomplished within 30seconds (Chandler and Gutierrez, 1986). The absorbed water mass in the filter paper was measured to the nearest 0.0001g using an analytical balance.

After the equilibrium between soil and filter paper, as it is an indirect measurement the soil suction can be obtained by an appropriate calibration curves (Noguchi et al., 2011). Different types of calibration equations are available depending on whether total suction or matric suction is measured; a single equilibrium equation is also available. The ASTM D 5298 proposed a single calibration equation to obtain both total and matric suction. Some authors have argued that the calibration curve should be different for matric and total suction (Houston et al., 1994; Leong et al., 2002). The filter paper water content was calculated using the calibration curve from Haghighi et al. (2012):

$$\mathrm{Ln}\,(\psi) = (a + b \times W_f + c \times T + d \times W_f \times T) / (1 + f \times W_f + g \times T + h \times W_f \times T) \qquad (3.2)$$

where ψ is soil suction (in kPa), W_f is the filter paper water content (%), T is temperature in Kelvin, and a = 10.8616232, b = -0.0637635, c = -0.0405607, d = 0.00021864, f = 0.01908312, g = -0.0036483, h = -0.0000765 are constant parameters. There was no intent within this thesis to evaluate the calibration curve; the main focus was to determine the influence of suction on subgrade soil behaviour.

3.3.3 Pressure plate test

The pressure plate extractor (1500F1 model) was used in this thesis to obtain the water retention curves (WRC); it was a 15 bar extractor (1500kPa). In the pressure plate, suction is generally measured directly by the axis translation technique. The axis translation technique is used to control and impose matric suction on a sample and was first proposed by Hilf (1956). It is the most common and widely used technique for controlling matric suction in the laboratory, when carrying out for unsaturated soil experiments (Matyas and Radhakrishna, 1968; Rampino et al., 1999; Toll and Ong, 2003; Farouk et al., 2004; Estabragh and Javadi, 2008; Ng and Tse, 2008; Yang et al., 2008; Uchaipichat and Khalili, 2009; Kasangaki, 2012). The basic technique entails is placing a soil sample on top of a saturated high air-entry ceramic disc, the base of which is connected with the pore water measuring system (Delage et al., 2008). The ceramic disc should have a higher air-entry value than the soil sample matric suction to avoid entry of the air bubbles into the measuring system. Figure 3.9 presents a schematic diagram of the setup of the pressure plate technique.

The soil specimen's dimensions were 50mm in diameter and 20mm high; the specimen was compacted in an oedometer cell to achieve the desired void ratio. It was decided from the GRAFT data that the sample would be prepared with 0.91 void ratio and 35% moisture content. The samples were prepared carefully to achieve the desire void ratio; oven dried soil was mixed with distilled water. Despite precautions, some of the samples actual void ratios varied slightly (±1.17%).

The prepared soil specimens were placed on top of a saturated ceramic disc. After that the target pore pressures were imposed and allowed adequate time to achieve equilibrium. The water pressure was applied through a GDS digital pressure\volume controller which was connected with a high air entry value (HAEV) ceramic disc and the air pressure was applied by a compressed air system. The laboratory air compression facility only allows a maximum value of 700kPa, therefore, in this test the maximum air pressure applied was 650kPa. The equilibrium period was considered 14 days based on the water movement in the GDS.

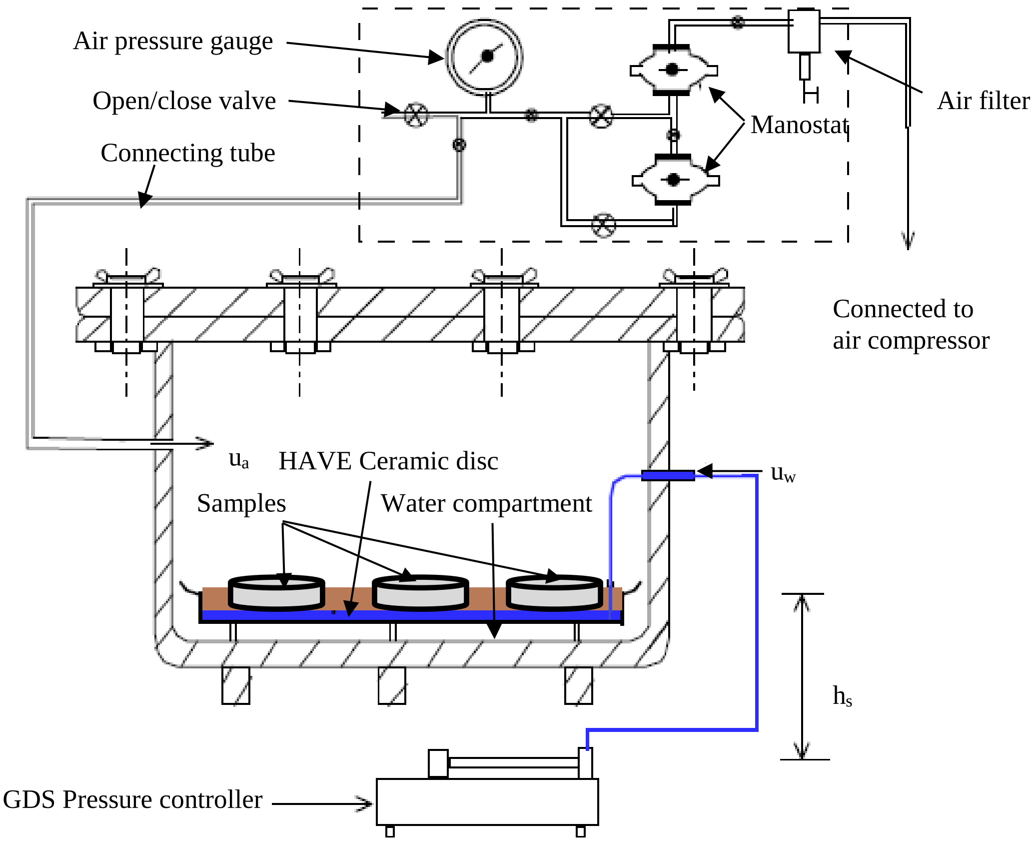


Figure 3.9 Pressure plate diagram for obtaining WRC (after Kasangaki, 2012)

In this research, the water retention curves were determined by the filter paper and pressure plate method. The main purpose of the water retention curve was to explain the subgrade soil behaviour resulting from flooding and the influence of cyclic wetting and drying effect on subgrade behaviour. Section 6.3.1 explains the water retention curves.

3.3.4 Oedometer test

The oedometer test is a dry or wet, stress-controlled confined compression or consolidation test. The collapse behaviour of subgrade soil has been studied in this research using double and single oedometer tests. The double oedometer test, which was initially proposed by Jennings and Knight (1957), was used to investigate the volume changes due to the wetting of soil. Cerato et al. (2009) reported that most compacted clay soils experience swelling, (increase in volume) under low confining stress and collapse behaviour (decrease in volume) under high confining stress. They also observed that all soils are sensitive to wetting, which induced collapse when soil experiences wetting under sufficient confining pressure.

The double oedometer test involves testing two identical soil samples. One of the soil samples is tested at natural water content. Another sample is initially saturated under a small seating load (i.e. 5kPa) and allowed to collapse. After that, both soil samples are loaded using the standard incremental method with the load increments the same for both samples. The difference of stress-strain behaviour between saturated and unsaturated samples indicates the collapse potential of the sample. Figure 3.10 shows the difference between a constant water content curve and fully saturated curve.

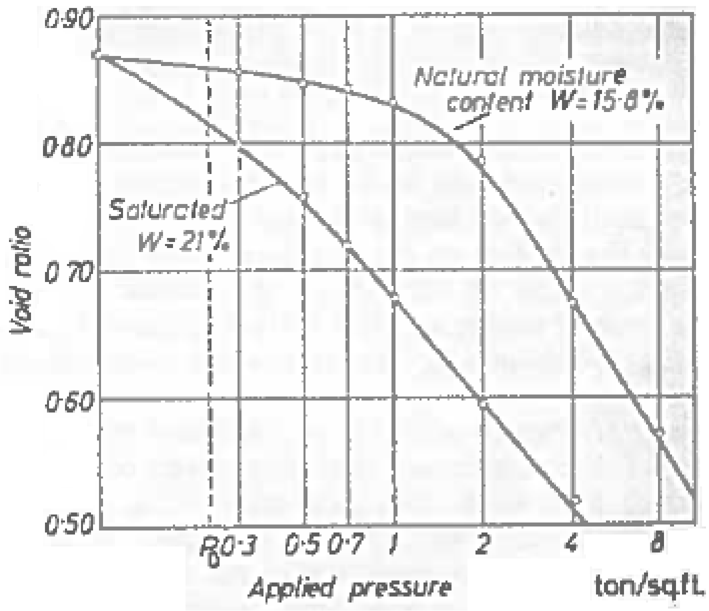


Figure 3.10 Illustration of a double oedometer experiment on hydro-collapsible soil behaviour (after Jennings and Knight, 1957)

The soil samples for this test were collected from the GRAFT (Figure 3.11). The sample was collected in three different stages, as presented in Table 3.3 After placing the two identical undisturbed soil samples into the rings of the two oedometers, the samples were kept under a pressure of 5kPa for 24 hours. One of the samples was then flooded whilst the other remained at its natural moisture content; the sample was then loaded according to standard incremental loading. The stress level was between 5kPa to 400kPa.The collapse potential for the soil samples was obtained from the following equation (Jennings and Knight, 1957):

$$\text{Collapse potential} = \frac{e_i - e_f}{1 + e_0} \qquad (3.3)$$

where e_i and e_f are the values of void ratio which are found from the oedometer curves at natural water content and at saturation condition respectively under the same applied vertical stress, e_0 refers to initial void ratio.

Figure 3.11 Collected soil samples for the oedometer test from the GRAFT

A single-point oedometer test was also conducted. The test was carried out according to ASTM D 5333. In this test, the natural water content soil sample was incrementally loaded until it reached the desired vertical stress (200kPa); then the sample was flooded. The comparison of stress-strain behaviour due to wetting under applied vertical stress was the collapse potential of the sample.

Table 3.3 List of oedometer test stages

Testing stage	Type of test
Before flooding	Single and Double
After 1st flooding	Single and Double
After 3rd flooding	Double

3.4. Full-scale investigation of track behaviour

In the railway structure there are so many uncertain and unpredictable things occuring which are not identified easily in normal laboratory test. Railway track design is not new; Sattler et al. (1989) reported that adequate railway track design developed over a long period of time; in fact railways were built before the advent of modern soil mechanics, but design and analysis yet remains more empirical from a geotechnical point of view. It is difficult to control test variables and to collect data on site. Therefore, a full-scale testing facility is necessary to investigate railway track behaviour under realistic conditions. Full-scale testing has advantages in terms of the ability to simulate long time-series events. Such testing forms an essential connection between laboratory work and theory and the site situation. The railway track structure is a complicated interaction between different elements such that the failure of one or more components can directly or indirectly lead to a series of undesirable consequences. In a small-scale, being able to simulate the complex events such as the effect of repeated flooding on track performance, cannot be achieved without changing subgrade. A review of full-scale testing facilities is presented below.

The Railway Test Facility (RTF) was designed and developed to produce dynamic loading and tamping cycles (Brown et al., 2007). Three actuators were used to apply traffic loading through three sleepers onto the ballast. A tamping bank, which was modified from a real tamper, was used for tamping. The testing facility was built in a concrete pit with dimensions of 2.1m (width) × 4.1m (length) × 1.9m (depth). Figure 3.13 presented a simulated traffic loading on the sleepers, which was achieved by applying sinusoidal loading up to 94 kN with a 90° phase lag between each actuator.

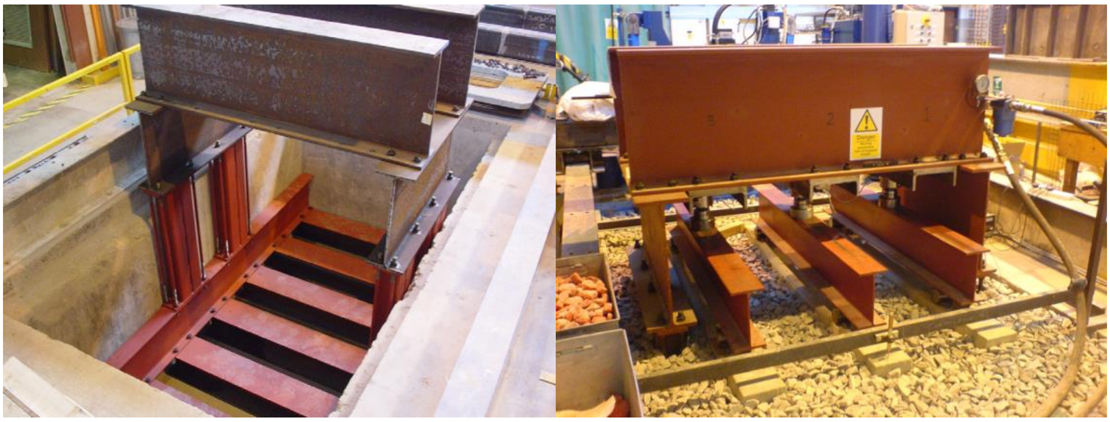


Figure 3.12 Railway testing facility (RTF) in University of Nottingham (Aursudkij, 2007)

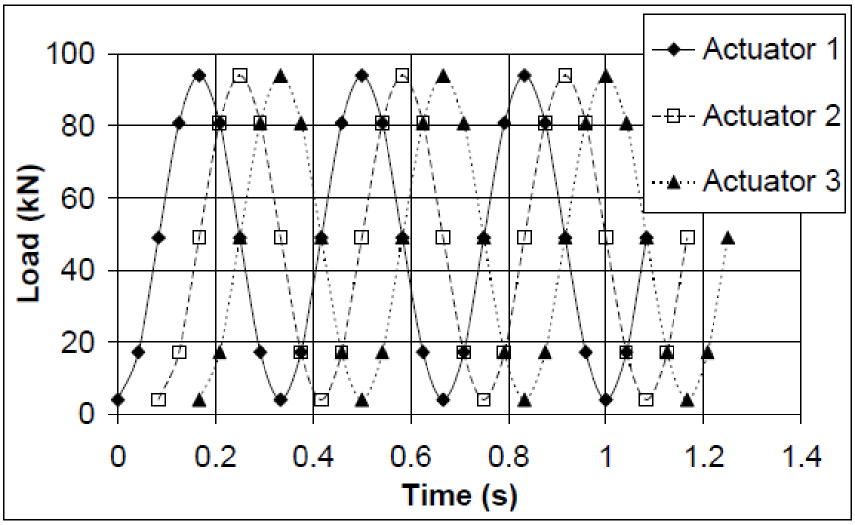


Figure 3.13 Loading pattern in the RTF (Aursudkij, 2007)

Al Shaer et al. (2008) also investigated in a full-scale to investigate the dynamic behavior and settlement of ballasted tracks (Figure 3.14). The experiment studied variables such as displacements, accelerations, pressures and settlements that allowed to better understand the dynamic behavior of a portion of a ballasted railway track, in order to estimate the settlement versus the number of load cycles.

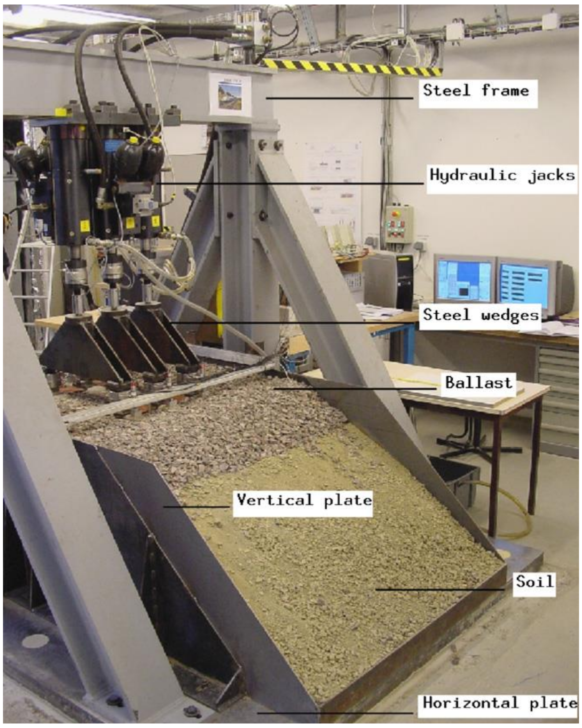


Figure 3.14 An experimental setup of a ballasted railway track with three sleepers (Al Shaer et al., 2008)

3.4.1 GRAFT testing facility

The Geopavement & Railway Accelerated Fatigue Testing (GRAFT) facility at Heriot-Watt University enables the testing of full-scale railway tracks under realistic railway loading conditions. A series of tests was performed in the GRAFT to investigate the impact of flooding on track performance. The previous researcher (Kennedy, 2010) using the GRAFT facility, investigated the influence of subgrade Young's modulus, applied vertical load and number of applied cycles on track settlement (included the rate of loading, mixed loading and ballast depth and different geosynthetics product). However, the research did not focus on soil behaviour particularly severe conditions such as flooding. A track settlement model was proposed, based on subgrade modulus which is unable to predict the track settlement during wet conditions. The research also did not distinguish the soil behaviour in unsaturated and saturated condition. A detailed discussion of this issue is presented in Chapter 6.

A cross section of the GRAFT can be seen in Figure 3.15. The track was constructed with three half sleepers and I -section steel beam which has similar stiffness properties to a BS 113A rail section, as presented in Table 3.4 (Kennedy, 2010). The I-beam was more suitable for the setup of load cell and LVDT in comparison with rail, therefore, the I-beam was used in the experiment.

Table 3.4 Comparison of I-Beam and Rail

Rail section	E (Young's Modulus N/m^2)	I (second moment of area m^4)	EI (bending stiffness Nm^2)
BS 113A	2.10×10^{11}	2.349×10^{-5}	4.933×10^{6}
GRAFT I-Beam	2.05×10^{11}	2.210×10^{-5}	4.531×10^{6}

Generally, the GRAFT consists of a track constructed within a steel tank, the dimensions of the tank are 1.072m wide x 3.0m long x 1.15m high. The Losenhausen UPS200 (LOS) machine is a closed loop control hydraulic machine, which operates from two pumps and has a 200 ton maximum capacity but it can be applied cyclically at 150T. The response of load or displacement of the hydraulic actuator is controlled by a servo valve which reacts to an electrical signal command to deliver oil pressure and flow up to a certain level to match the signal (Woodward et al., 2009). The tank is supported laterally from four 50mm×50mm steel angles around the top of the tank and two 127× 64×14.9mm channel sections welded continuously around the tank at 200mm and 500mm from the base of the tank (Kennedy, 2010). Figure 3.16 presents a prepared track under LOS.

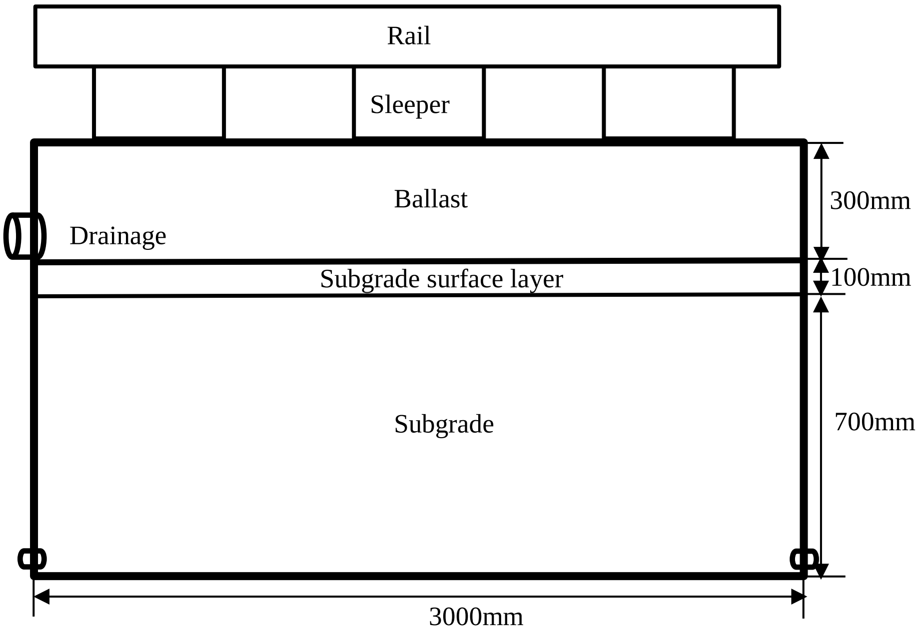


Figure 3.15 Cross-section of GRAFT

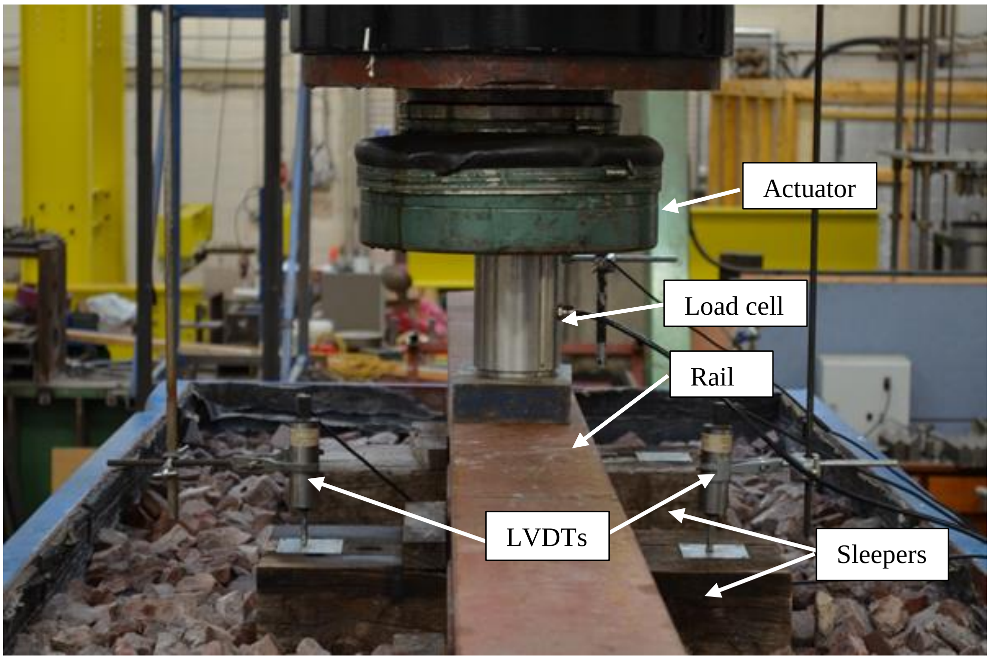


Figure 3.16 Shows full scale testing facility GRAFT track under LOS & instrumentation

The GRAFT load was considered to be 90kN calculated from the following equation (Kennedy, 2010). Li et al (2007) also used the following relationship. Kennedy (2010) also reported that 40,000 cycles in GRAFT at 90kN is equivalent of 1million gross tonnes (MGT) of applied load in real life.

Applied load in test = Wheel load × Sleeper load factor × Load area stress factor × Dynamic load factor (3.4)

The axle load was 25 tonnes as this is the maximum load permitted on UK track. The sleeper load factor was accounted for at 85% due to the reduced load distribution as three sleepers were used; full 100% load distribution is found in 5 sleepers (Profillidis, 2006) as shown in Figure 3.17. The load area stress factor was 35% calculated from the deflection profile along a sleeper on the ballast surface based on the work of Selig and Waters (1994). The dynamic load factor was 120% (Kennedy et al., 2012). The axle load used here was only a guide because the exact load depends on several factors, such as type of sleeper, spacing and dimensions, subgrade quality etc. (Selig and Waters, 1994; Profillidis, 2006). The applied load is:

Applied load = 250kN×85%×35%×120%
= 89kN

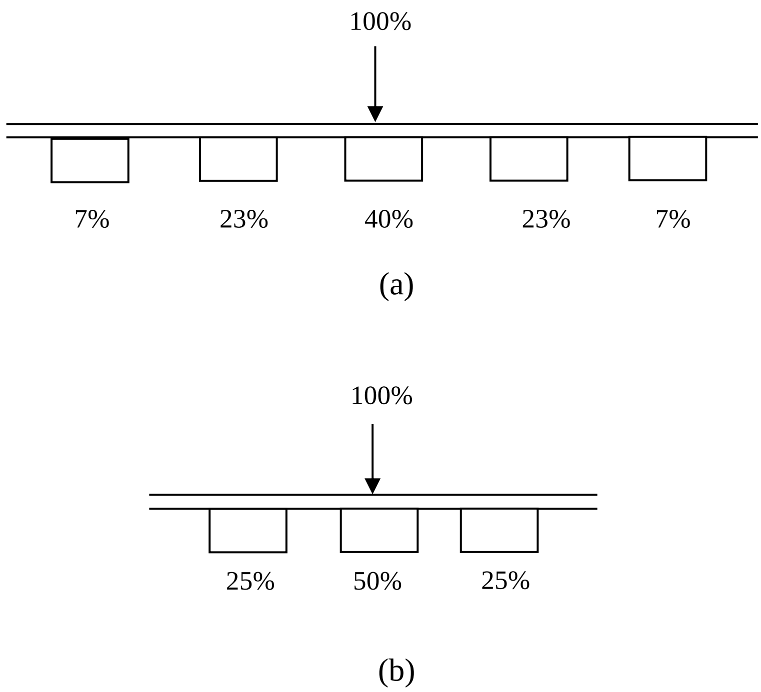


Figure 3.17 Load distribution of axle along the sleepers (a) suggested by Profillidis (2006) and (b) suggested by Awoleye (1993) and used in this project

3.4.1.1 Instrument calibration

Two 100 tonne load cells were used in this research. Load cells were calibrated from a 50 tonne capacity Denison monotonic testing machine. The load cell and Denison voltage were connected to a computer through a USB connected input board. The voltage outputs from both calibrated Denison and load-cell instrumentation were recorded simultaneously by software DaqView. The equations of the best-fit straight lines for the two 100 tonne capacity load cells and the relation between converted voltage in kN presented in Figure 3.18. The load cell CCDG was used during track loading on track and DSCC was used for the plate load test.

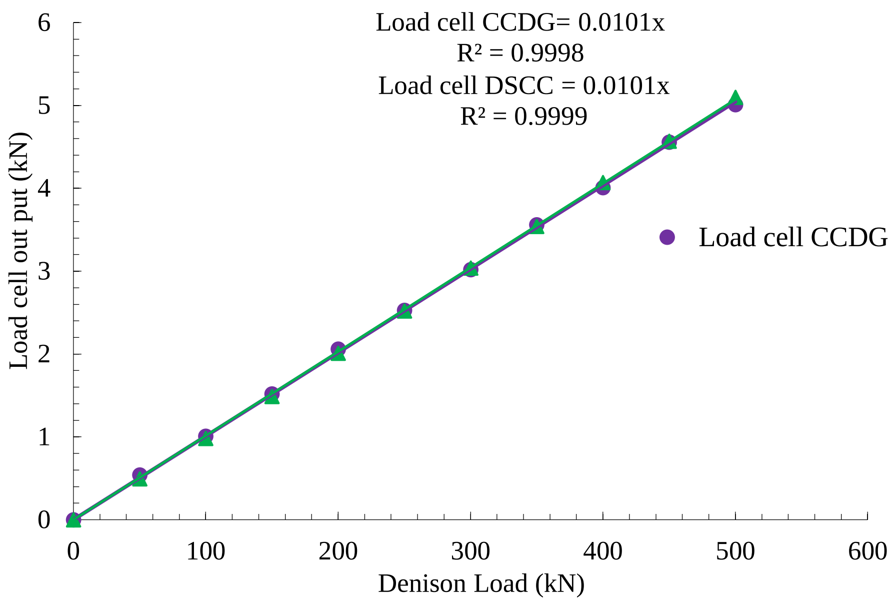


Figure 3.18 Calibrated load cells against Denison load

The displacement of the track was measured individually by three LVDT. Two Positek LVDT 15327 and 15328 and one HLP 190 LVDT were calibrated by micrometer from volt to millimetre. Figure 3.19 shows the calibration results.

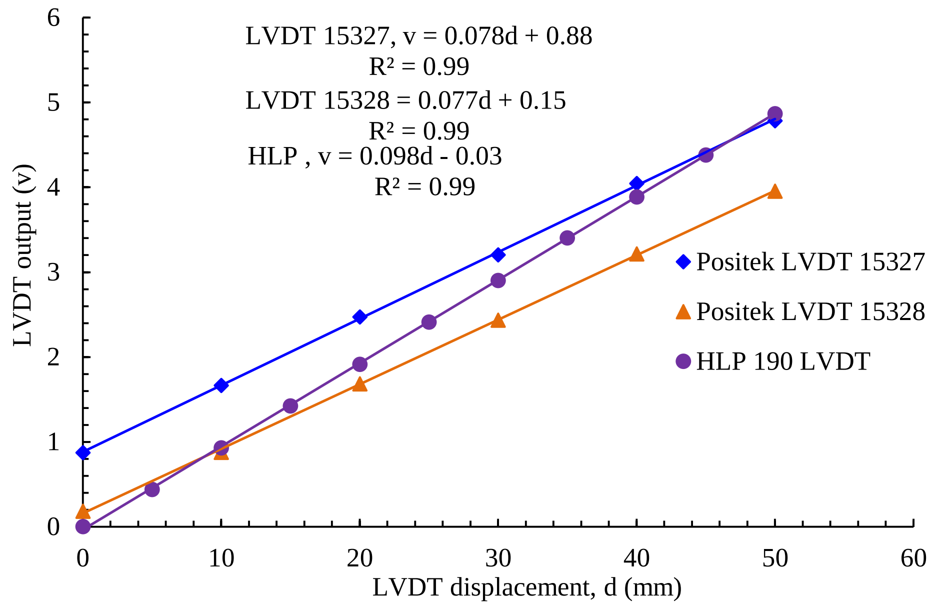


Figure 3.19 Calibration curves for LVDT's

3.4.1.2 Limitations of the GRAFT

The LOS machine is limited to only one actuator; therefore, it can only apply vertical compressive loads in one position. The load distribution from both sides of a centrally loaded sleeper from a rolling wheel was not considered. To realistically monitor the centre sleeper response to a rolling wheel at least three sleepers are required (Kennedy, 2010); therefore the loading mechanism in the GRAFT replicates a repeated quasi static single wheel load on a central sleeper. The effects of principal stress rotation were also not considered. The investigation was based on the middle sleeper only. In order to limit the confinement effects from the walls of the tank to the substructure and to provide lateral support similar to the horizontal residual support experienced in the field, the tank was lined with 12mm thick neoprene rubber. Neoprene was chosen for its low stiffness and high resistance to abrasion.

3.4.2 Track preparation

The track was constructed with a 700mm kaolin clay subgrade layer, overlaid by a kaolin clay formation layer (100mm) and overlain by a 300mm ballast layer included three, half hardwood sleepers overlain by a rail (I-section steel beam). The track and compacted soil was inherited from a previous experiment (more than a year ago). The purpose of keeping the subgrade soil was to investigate the track performance on an operational track. If the subgrade was replaced, then it would have been considered as a new track which cannot simulate the operational track. It was mentioned earlier that a newly constructed track experienced significant track settlement from the subgrade, as the newly constructed track never have experienced traffic load.

After removing the ballast, soil samples were collected at predetermined depths and locations, primarily under the three sleepers, in order to evaluate the soil properties. The soil properties included moisture content, void ratio and soil suction (matric and total suction). Three samples were also collected for oedometer testing to investigate the collapse behaviour. To check the soil properties subgrade was extracted at different locations and depths. The replaced soil was compacted to achieve a similar density (1.54Mg/m3) first by an electric compactor (40kPa) then placed under the LOS with a cyclic load applied. The tank was divided in four sections and each section was compacted under a 100kPa for 1000 cycles. Figure 3.20 presents the compacted subgrade soil.

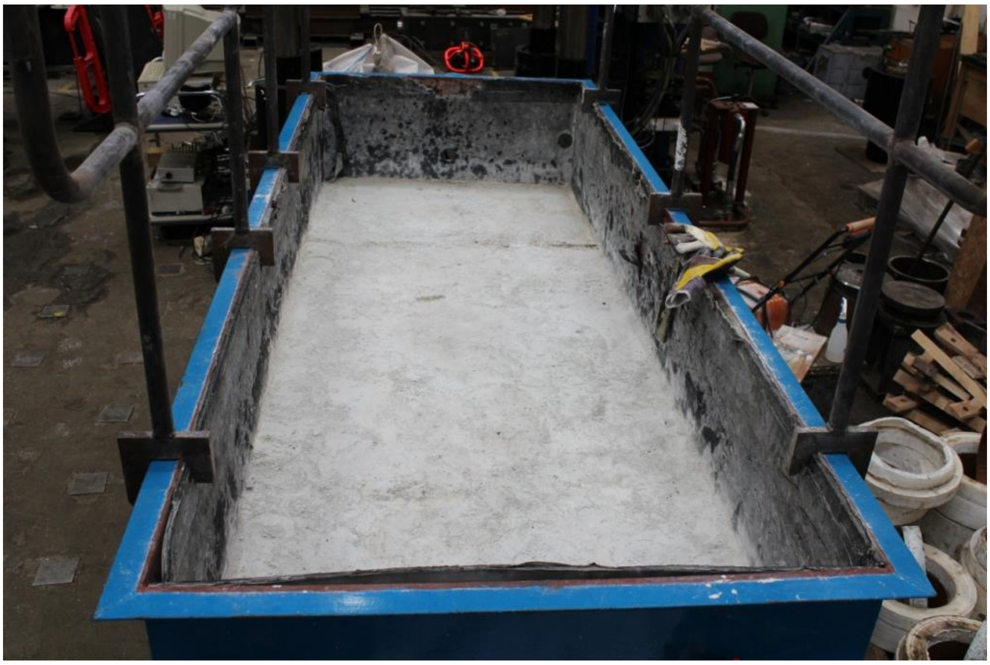


Figure 3.20 Compacted subgrade soil

A 300mm ballast layer was placed on top of the subgrade soil which is typical of ballast depth in the UK (RT/CE/S/006, 2002). The ballast placement is shown in Figure 3.21. The ballast was compacted in three, 100mm layers by an electrical compactor to achieve 1.60Mg/m^3. In this research, sub-ballast was not used prior to an investigation of the ballast and subgrade soil behaviour. Figure 3.22 shows the positioning of prepared track under the LOS. A prepared track can be seen in Figure 3.23 after placing an I-section beam on top of the ballast.

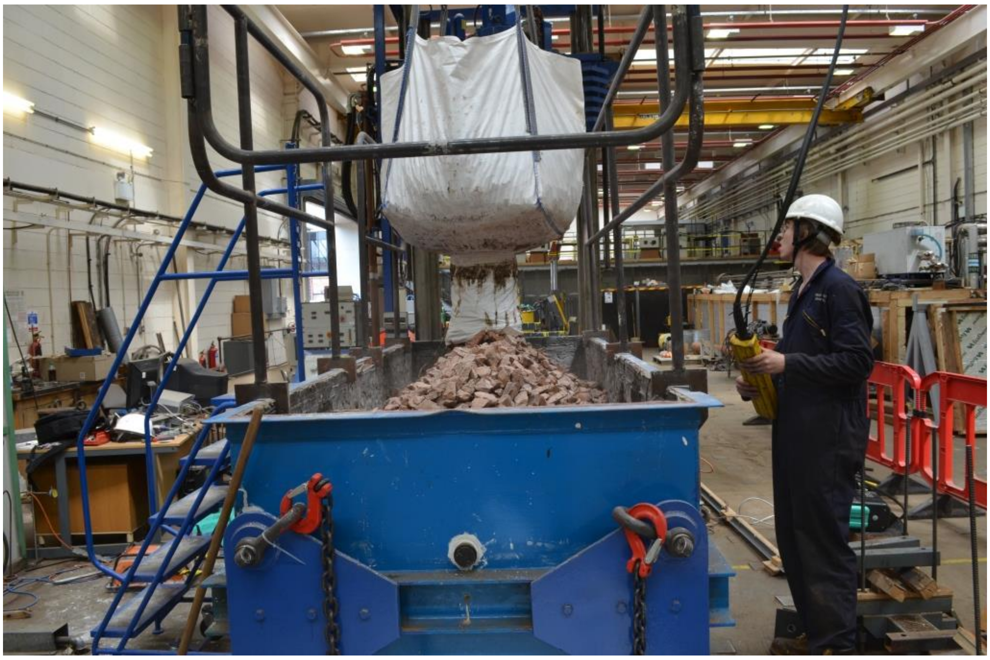


Figure 3.21 Ballast placements on top of subgrade

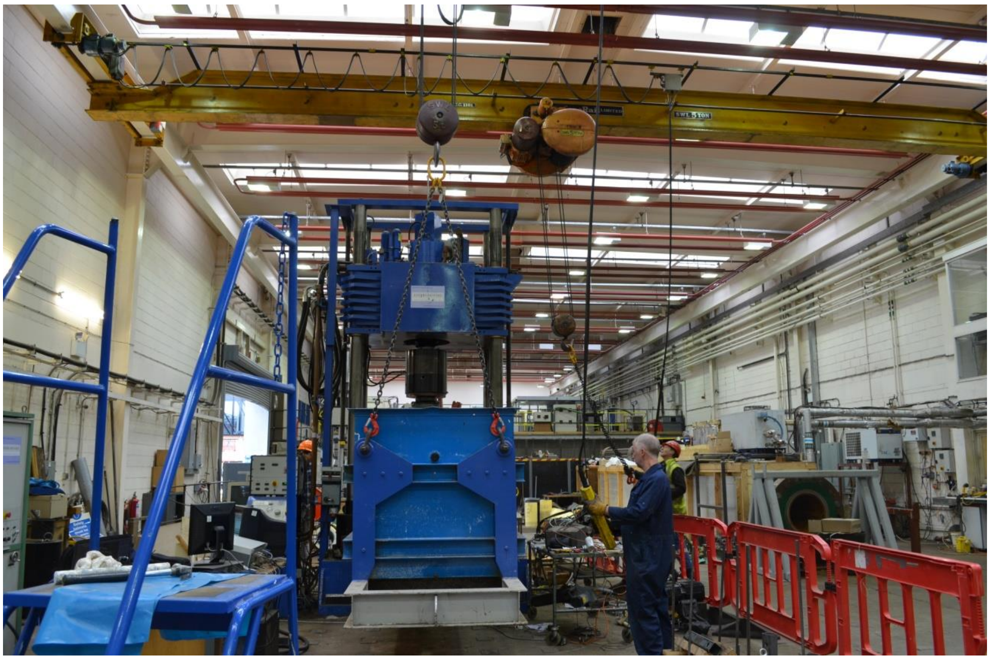


Figure 3.22 lifting processes of the tank

Figure 3.23 A full scale prepared track before test

Before the start of the experiment, two Positek LVDT's 1327 and 1328 were placed on either side of the middle sleeper to measure the displacement. The load cell was placed under the actuator on top of the middle sleeper. All the LVDTs and the load cell were connected to a data logging system which was connected to a desktop computer. DaqView software was used for data recording. All the data were recorded at 30Hz.

3.4.3 GRAFT testing programme

The testing programme in GRAFT was divided in two. Experiment-1 was further subdivided in three phases: a) Phase-I: initial pre-flooding phase, b) Phase-II: after 1st flooding and c) Phase-III: Recovery period. Experiment-2 was also subdivided in three phases: a) Phase-I: after 2nd flooding (with a 150mm sand blanketing), b) Phase-II: after 3rd flooding (with a 150mm sand blanketing and without any water being drained) and c) Phase-III: New surface layer (dry). A time plan is presented in Figure 3.24.

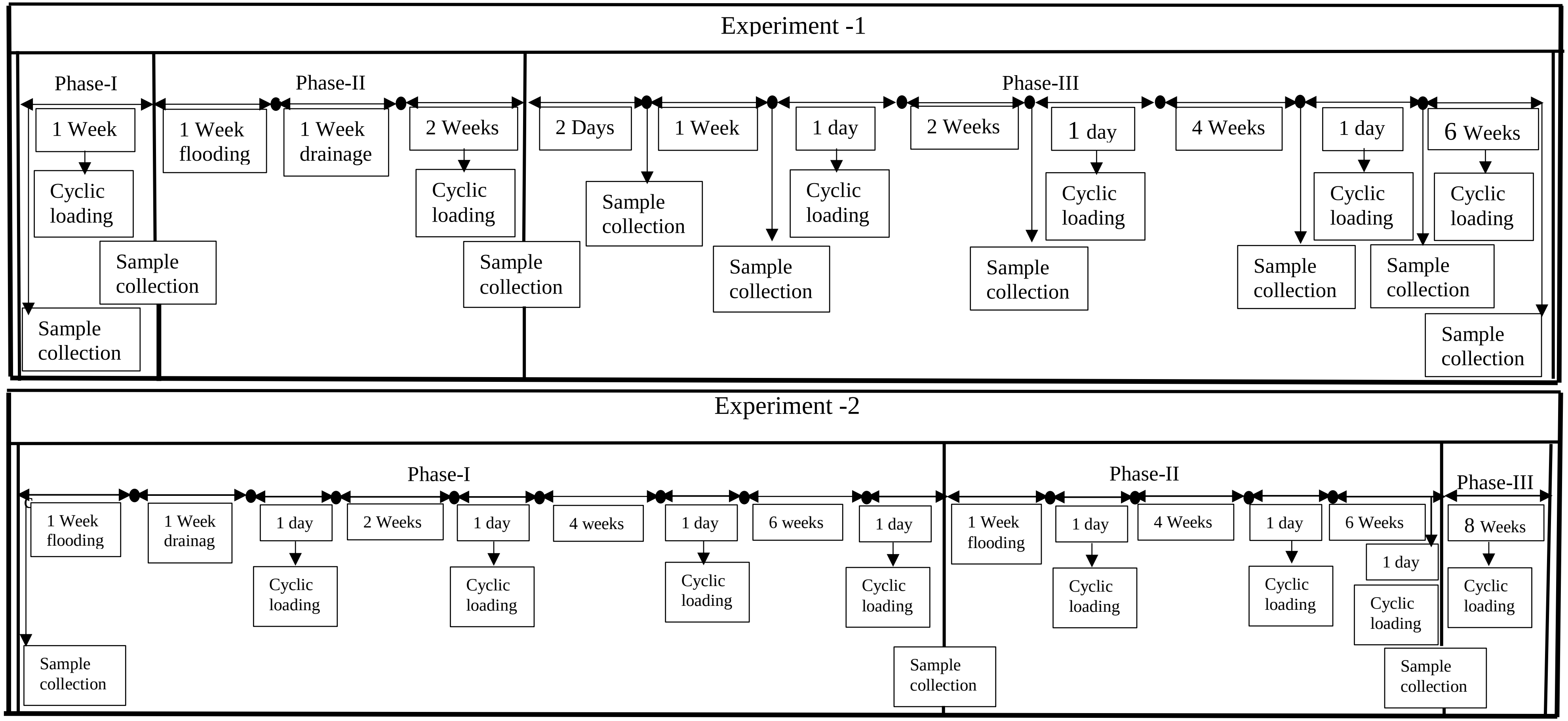


Figure 3.24 Time plan of the experimental programme

3.4.3.1 Experiment-1

Phase-I: initial phase (dry)

At the beginning in the first test (before flooding), the applied loading frequency in the GRAFT facility was 3Hz at 90kN. Generally, the frequency of the traffic loading typically varies from 1Hz to 3 Hz (Ghataora et al., 2004); however, the frequency is dependent on various factors such as speed, the axle, bogie and coach spacing etc. In addition, 3Hz at 90kN is the maximum capacity of the GRAFT facility. During Phase-I, 5×10^5 load cycles were applied.

Phase-II: after 1st flooding (wet)

In the second Phase (after flooding), the surface layer became significantly soft; therefore, the loading frequency was reduced from 3Hz to 2Hz. The total applied cyclic load at this phase was 2.3×10^5 cycles. In this Phase, frequent tamping maintenance had to be undertaken, the details of which are discussed in section 4.4.1.

Phase-III: recovery period (dry)

Phase-III was considered as a drying Phase or recovery period after flooding. This Phase is very sensitive to loading as the subgrade regains its strength and if proper action is not taken, it can damage the track permanently. The track was allowed to dry, two days later soil properties were checked but due to unacceptably high moisture content and low matric suction, no further loads were applied. Soil properties were measured again after one week but no significant improvement was perceived. After another two weeks, 20,000 cycles were applied but the settlement was exceptionally high. Four weeks later a further 20,000 cycles and six weeks later another 20,000 cycles were applied. The results are discussed in section 4.5.

3.4.3.2 Experiment-2

Phase-I: after 2nd flooding (with sand blanketing)

The first Phase of Experiment-2 was conducted with a 150mm sand-blanket (Network Rail standard, RT/CE/S/033) overlying the subgrade soil, in order to investigate the influence of sand blanketing on track behaviour during and after flooding. In this Phase, subgrade soil was not modified except for making the surface layer level. It was then allowed for two weeks for air-drying. The track was flooded for second time for a week. After the draining stage completed, the tank was placed immediately under the LOS to investigate the influence of sand blanketing on track performance. The drainage period was one week; it was ensured that no water coming out from the drainage holes. At this Phase, only 1400 cycles were applied at 2Hz. The test was repeated after two (2000cycles), four (10,000cycles) and six (10,000cycles) weeks.

Phase-II: after 3rd flooding (with sand blanketing and water inside the tank)

The test was conducted with a 150mm sand blanket (Network Rail standard, RT/CE/S/033) and with water inside the tank. It was found in Phase-I that the sand blanketing protected the subgrade from subgrade erosion, slurry formation and ballast movement into the soil. However, the sand blanket caused additional problems such as water becoming entrapped between ballast and the sand blanket layer, as well as sand migration etc. The track was marked rectangular to investigate the ballast movement during cyclic loading under water. After one week of flooding, the track was placed under LOS without any water being drained away. The track submerged rapidly after only 1500 cycles. The initial loading frequency was 2Hz, which was reduced to 1Hz. The test was repeated after four weeks (at 2,000 cycles) and six (at 10,000 cycles) weeks.

Phase-III: New surface layer (dry)

The test was conducted with a new surface layer considered as a dry phase to compare the track performance with the initial dry Phase. The surface layer became saturated due to repeated flooding; therefore, it was decided to replace the surface layer. In addition, the bottom layer of the tank also became saturated water passed through the layer via the side wall of the tank. The problems due to a raised of water table was studied in this Phase. After removing the ballast, the surface layer (100mm) was also removed and replaced with a new soil layer. The soil was mixed with approximately 12% moisture content in a big mixture machine. The reason for mixing in low moisture was as the remaining subgrades moisture content was considerably higher. When the soil was placed on top of the subgrade it would be easy to compact. The soil was placed in four layers and each layer was compacted by both manually and an electric compactor. The tank was then placed under the LOS to compact the new surface layer. The compacting procedure under the LOS followed the same procedure as discussed in section 3.4.2. After compaction, the track was allowed to air dry for 8 weeks, as the test was required to run in dry conditions. Before placing ballast, soil samples were collected to measure the soil properties (moisture content, void ratio and suction). The moisture content was 12% and matric suction was approximately 700kPa. The applied cycle was 2.3×10^5 at 3Hz. The behaviour of the track is discussed in section 5.4.

3.5 Summary

In this chapter, the experimental techniques and procedures have been presented. The cyclic load was applied in the GRAFT in unsaturated and saturated conditions and the subgrade soil behaviour was investigated separately. Soil suction was measured by the filter paper method and WRC determined by both filter paper and pressure plate techniques. Double and single point methods were used to investigate the collapse behaviour of subgrade soil. Subgrade stiffness was determined by the PLT and the relationship between suction and stiffness was studied. A range of tests of ballast characteristics tests was conducted including a particle size distribution test and the large shear box test. The main objective of the experimental programme was to obtain reliable and realistic data that can be used in track design and maintenance recommendations. Table 3.4 presents a summary of all the performed experiments and conditions of the test in this research. The following chapters explain the results from the experimental programme that was designed to fulfil the stated objectives of this research.

Table 3.5 List of all experiments performed in this experiment

Tests	**Condition of test**	**Performed test**	**Applied cycles**
Experiment-1		LDSB, PSD and PP	
Phase-I	Dry: Initial	PLT, WC, FP, OM and GRAFT	500,000
Phase-II	Wet: 1st flooding	WC, FP and GRAFT	230,000
Phase-III	Dry: Recovery period	WC, FP, GRAFT, OM and PLT	50,000
Experiment-2		LDSB, PSD and PP	
Phase-I	Wet: 2nd flooding (with sand blanket)	PLT, WC, FP, and GRAFT	13,400
Phase-II	Wet:3rd flooding (with water inside and sand blanket)	PLT, WC, FP, OM and GRAFT	11,500
Phase-III	Dry: New surface layer	PLT, WC, FP and GRAFT	300,000

LDSB = Large Direct Shear Box, PSD = Particle Size Distribution, WC = Water Content, PLT = Plate load test, FP = Filter Paper, PP = Pressure Plate and OM = Oedometer test

CHAPTER FOUR - IMPACT OF FLOODING ON TRACK PERFORMANCE

4.1 General overview

This chapter presents details of an experimental study of railway track behaviour both before and after immediately flooding and subsequently, after a period of drying. The railway structures are constructed on compacted soils (unsaturated soil). The strength and stiffness of unsaturated soil are greatly influenced by soil suction; therefore, design and construction measures should be implemented to maintain the unsaturated conditions throughout the service life (Siekmeier, 2011). It should be emphasised that the saturated condition is the critical situation that reduces long-term performance. Unsaturated soil behaviour being highly dependent on soil suction and so varying with the changes of water content (Fredlund and Rahardjo, 1993). In unsaturated soil mechanics, matric suction is a key parameter which controls the state of stress (Fredlund and Morgenstern, 1977; Fredlund and Rahardjo, 1993; Houlsby, 1997; Gupta et al., 2007; Sawangsuriya et al., 2008; Ng and Xu, 2012). Unsaturated soil behaviour has been highlighted in the literature review. In this chapter, the subgrade behaviour is explained in terms of changes in moisture content associated with matric suction.

To understand the influence of flooding on track performance, this part of the experimental programme was divided into three phases.

4.1.1 Phase-I (Dry-initial condition)

Initially, the subgrade soil was in a dry state (unsaturated), the moisture content and void ratio at surface layer (100mm) were approximately 10% and 0.91 respectively and the degree of saturation was 30%. The matric and total suction were, approximately, 1300kPa and 2000kPa respectively. The track settlement in this phase was significantly low due to the strong support from the subgrade. Track performance during Phase-I is discussed in section 4.3. Details of moisture content, void ratio and soil suction are presented in Table A.1 in Appendix A.

4.1.2 Phase-II (wet condition after flooding)

The track was flooded for a week up to the ballast level. The water was then drained (one-week drainage period) and the track was placed under the LOS immediately to investigate the track behaviour in the wet condition. The moisture content of surface layer was approximately 43%. Table A.2 (Appendix-A) presents the data at different depths within the subgrade soil including moisture content, void ratio and soil suction. The track behaviour during this phase is discussed in section 4.4.

4.1.3 Phase-III (drying period)

This stage is considered as the drying period or recovery period. The track performance and subgrade profile were checked at different time intervals (see Figure 3.24). After Phase-II was completed, the track was allowed to dry. The track performance was checked after two, four and six weeks to investigate the improvement of track performance. A summary of soil properties is presented in Table A.3 in Appendix A. The details of track performance during Phase-III are presented in section 4.5.

A full-scale test is shown in Figure 4.1 where at the beginning of the test, 2×10^5cycles were applied at a frequency of 3Hz at 90kN directly at the middle sleeper through the I-Section beam. After tamping the ballast, a further 3×10^5 cycles were applied to investigate the tamping maintenance effect in the GRAFT. Track preparation and experimental procedures are described in Chapter 3.

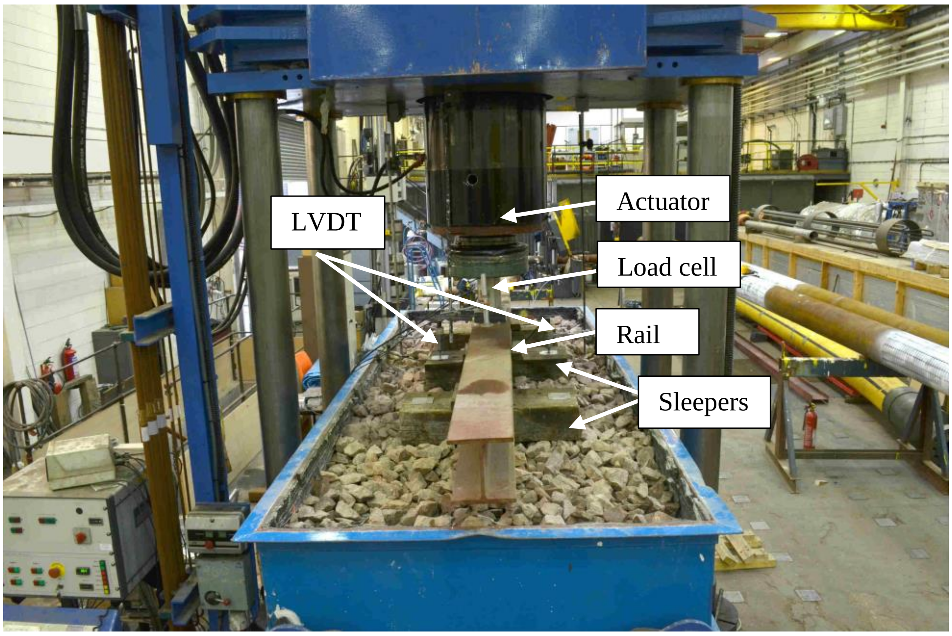


Figure 4.1 Track under the loading action

4.2 Track structure and performance

Conventional ballast track formation consists of the rail, ballast and sleepers. Usually, the ballast lies directly on the subgrade, depending on the bearing capacity of the soil layer. Sometimes a layer of sand blanketing or separator is place under the ballast to improve the drainage facility. The track performance depends on the behaviour of the underlying subgrade materials; track geometry manipulates the rail track performance. The track performance could be degraded if subgrade strength is reduced by ballast fouling or by subgrade attrition. Fouled ballast impedes water drainage facility where water could be trapped, which can cause further subgrade softening (Aw, 2007). High track stiffness can cause greater dynamic overloads on the rail with enhanced train-track interaction forces, where a low stiffness value of track causes a flexible track with poor energy dissipation (Pita et al., 2004). Berggren (2009) stated that the present understanding of track stiffness and impact of track performance is not sufficient. This applies particularly to track stiffness and soil suction as they are rarely investigated. Track settlement is a result of track deterioration from sub track aspect; differential track settlement considered as a result of faulty geometry which can cause derailment if not treated early or can increase maintenance costs (Kennedy, 2010).

Selig and Cantrell (2001) reported that substructure maintenance mainly focused on ballast and then second priorities are drainage ditches; the deterioration of the track components or the main cost of the track maintenance directly related to the drainage system. It is well documented that the excess water in railway infrastructure induces significant degradation in load carrying capacity. An effective drainage system can reduce maintenance and extend structural life. Post flood track behaviour presents many engineering challenges and their behaviour is very complicated due to changes of soil behaviour under cyclic loading.

4.3 Track settlement behaviour at Phase-I

Initially the subgrade soil was in an unsaturated state. The moisture content and void ratio at the surface layer (100mm) were approximately 10% and 0.91 respectively, and the degree of saturation was 30%. The matric and total suction were approximately 1300kPa and 2000kPa respectively. In other depths and locations in the GRAFT, soil suction (matric and total) was found to be less than at the surface layer. At a depth between 100-500mm, the moisture content was averaged 15% and the degree of saturation was approximately 50%. The matric and total suction were approximately 700-1500kPa respectively. Figure 4.2 shows the variation of moisture content, the degree of saturation, matric and total suction at the different depths under the middle sleeper.

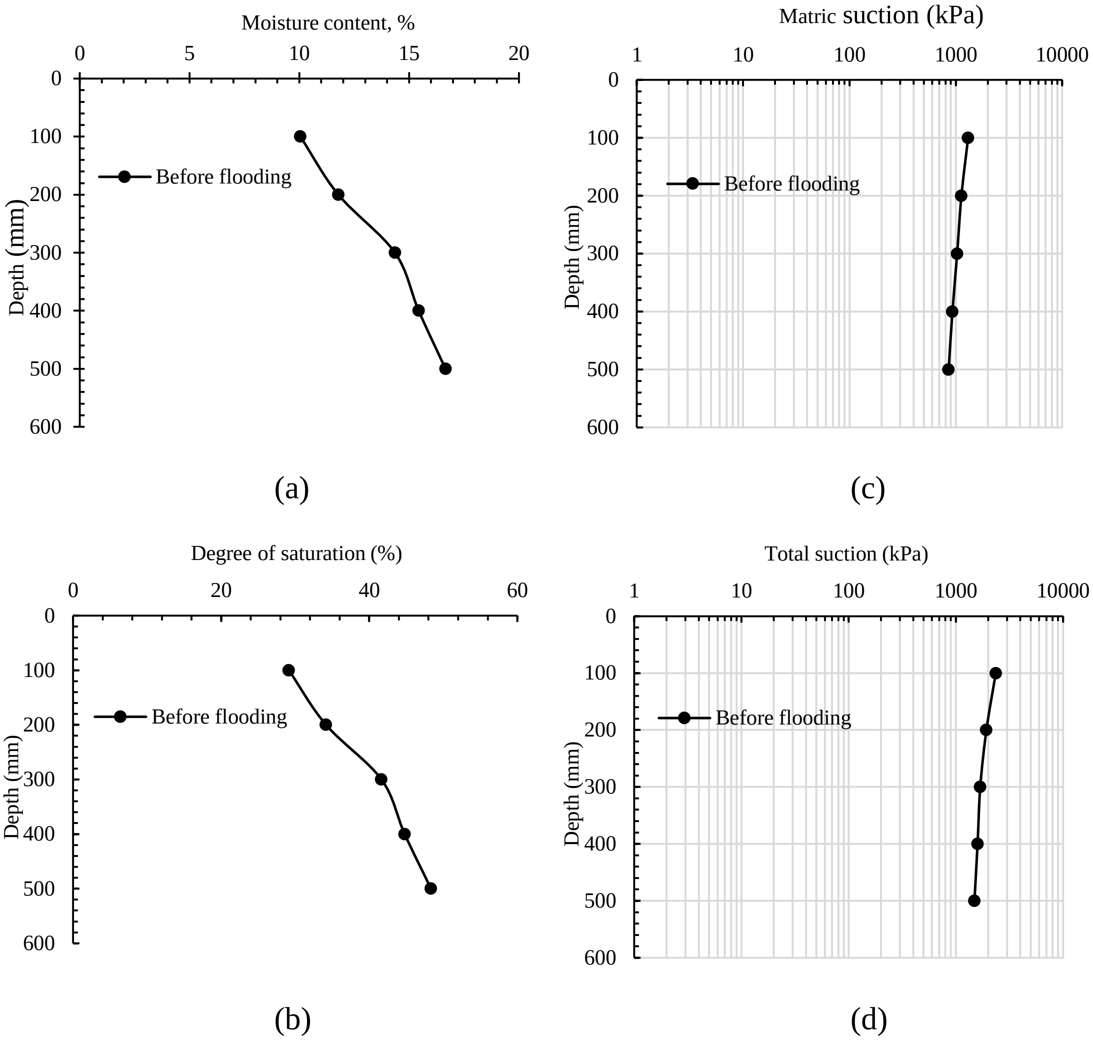


Figure 4.2 Variations of (a) moisture content, (b) degree of saturation, (c) matric suction and (d) total suction before flooding at different depth under the middle sleeper

The first test conducted in dry condition (unsaturated) in the GRAFT-I facility was to investigate the track performance in dry condition including the tamping maintenance effect. The track settlement behaviour before and after tamping is presented in Figures 4.3 and 4.4 respectively. The track settlement behaviour followed the same trend here as previous railway research work (Aursudkij, 2007; Ionescu, 2004; Kennedy, 2010; Ghataora et al., 2004). At the beginning of the test (Figure 4.3), the initial rapid track settlement (first stage settlement) was approximately 5mm due to initial ballast densification, which was followed by a second stage linear settlement with time/number of cycles (Dahlberg, 2001). After densification of ballast, the track settlement increased linearly, approximately 7mm at the end of 2×10^5 cycles. A theoretical prediction, Equation (2.7 and 2.8) for the GRAFT developed by Kennedy et al. (2012) is also plotted for a comparison. The experimental measurement (7.62mm) is 18% higher than prediction by the model (6.46mm). The model however, a brief discussion is presented in Chapter 6.

The subgrade showed a very stiff due to higher soil suction (1300kPa). Increasing soil suction causes on increase in the stiffness of unsaturated compacted soil as the matric suction generates an additional effective confining pressure in the soil structure (Mendoza and Colmenares, 2006). Gupta et al. (2007) stated that the capillary menisci among the particles create an additional inter particle force which causes the increase of modulus of contacts, that means the modulus of unsaturated soil particulate media depends on matric suction, the matric suction increases due to increases of surface tension forces or capillarity between particles. Lu and Likos (2004) stated that water remaining in the voids of unsaturated soils causes a very high negative pressure which creates tensile forces to increase the effective stress and bring the soil particles together. At higher suction, the sensitivity of stiffness to deviatoric stress increases (Dawson and Correia, 1996) therefore, a high-quality track performance was observed.

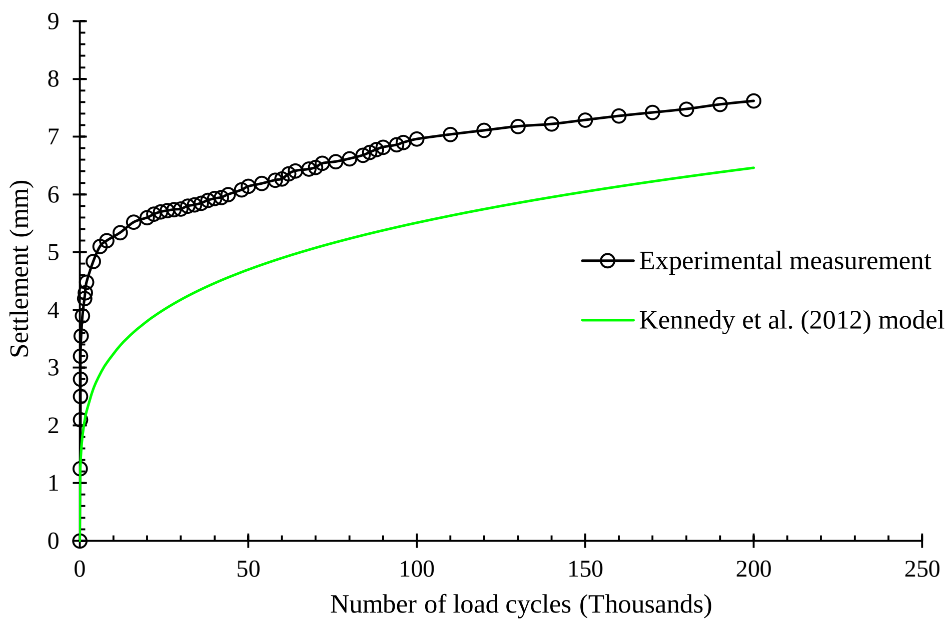


Figure 4.3 Track settlement during initial 2×10^5 cycles

The ballast was tamped under the sleepers after 2×10^5 cycles to bring the track back the up to the level prior to the tamping. After tamping the track, a further 3×10^5 cycles were applied at 90kN, the frequency of the cycles was 3Hz. The track settlement was 18% higher than the pre-tamping stage. Aursudkij (2007) and Selig and Waters (1994)also noted that the tamping induced a faster rate of initial track settlement. After completion, the track settlement was approximately 9mm. However, the settlement rate of the track was very slow (0.10mm/1000cycles) due to strong support from the subgrade.

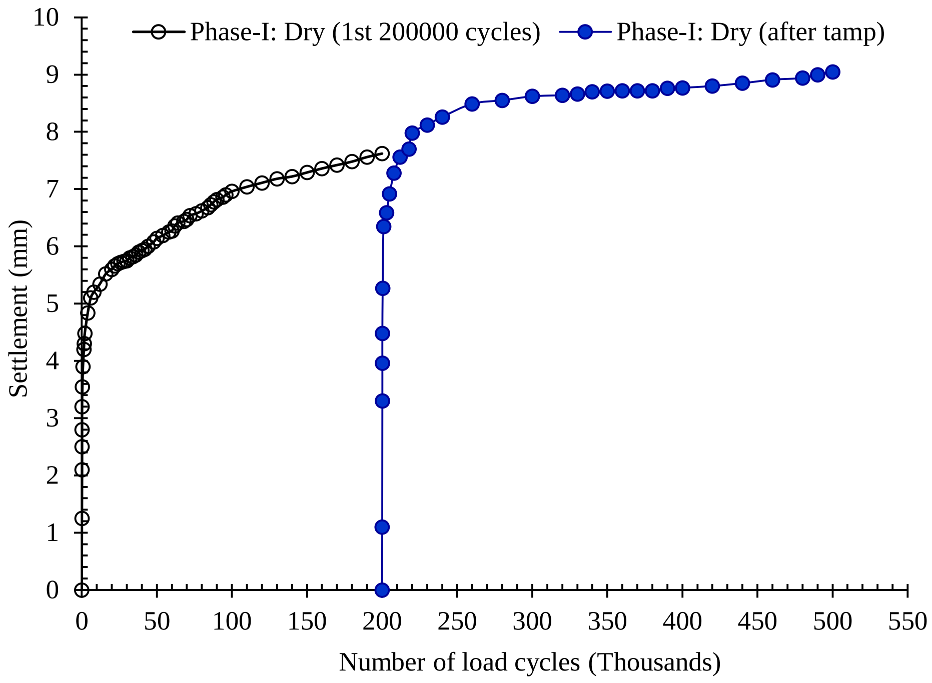


Figure 4.4 Track settlement behaviour at Phase-I including tamping effect

4.4 Track settlement behaviour at Phase-II

In order to investigate the track behaviour after flooding, the track was flooded for a week up to ballast level is shown in Figure 4.5. The GRAFT was flooded by a hose, the water was pouring at a slow speed and the hose was moving around the tank to avoid any kind of subgrade erosion.

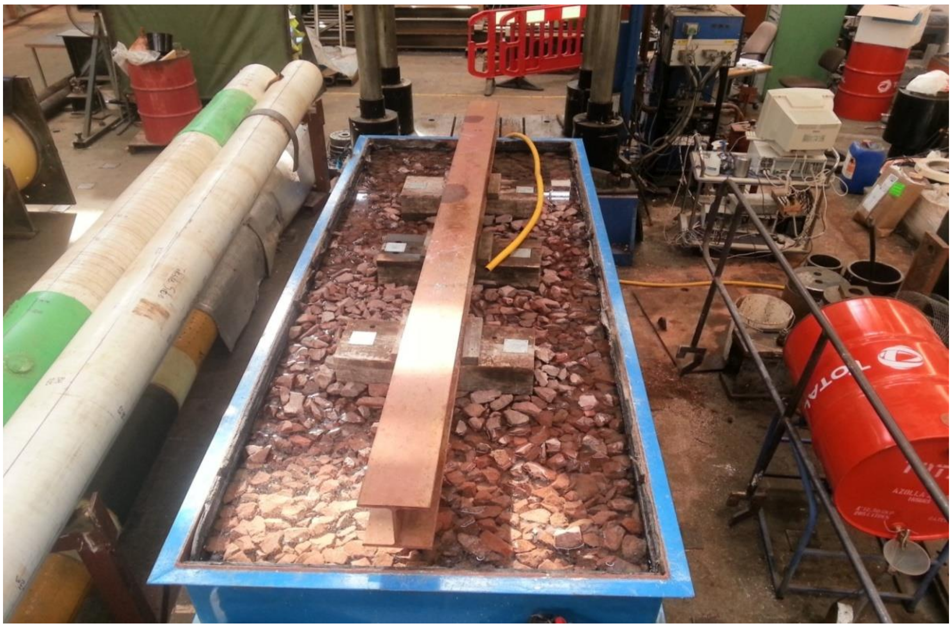


Figure 4.5 Track is flooded for a week

The following sections discuss the track behaviour immediately water drainage from the track and the subsequent drying period. In this phase, most the challenging aspect was sample collection without disturbing the track. After removing the rail, the ballast was carefully removed from under the middle sleeper and then a 200mm long and 100mm diameter tube was placed to prevent the ballast collapsing into the hole. A 100mm long and 20mm diameter pipe was then used to collect the soil sample. It was difficult to get a sample due to ballast penetration into the soil. The plate load test was not carried out at this point as the ballast was not removed.

The moisture content of subgrade surface layer was approximately 43% and the degree of saturation was 100% are shown in Figure 4.6 (a) and (b). The soil suction (matric and total) was found to be zero up to 200mm. The soil sample was collected over the 500mm under the middle sleeper. The moisture content was between 26-33% and the degree of saturation was between 88-100%. The matric suction was measured between 10-450kPa (Figure 4.6c) and the total suction was measured between 150-550kPa (Figure 4.6d).

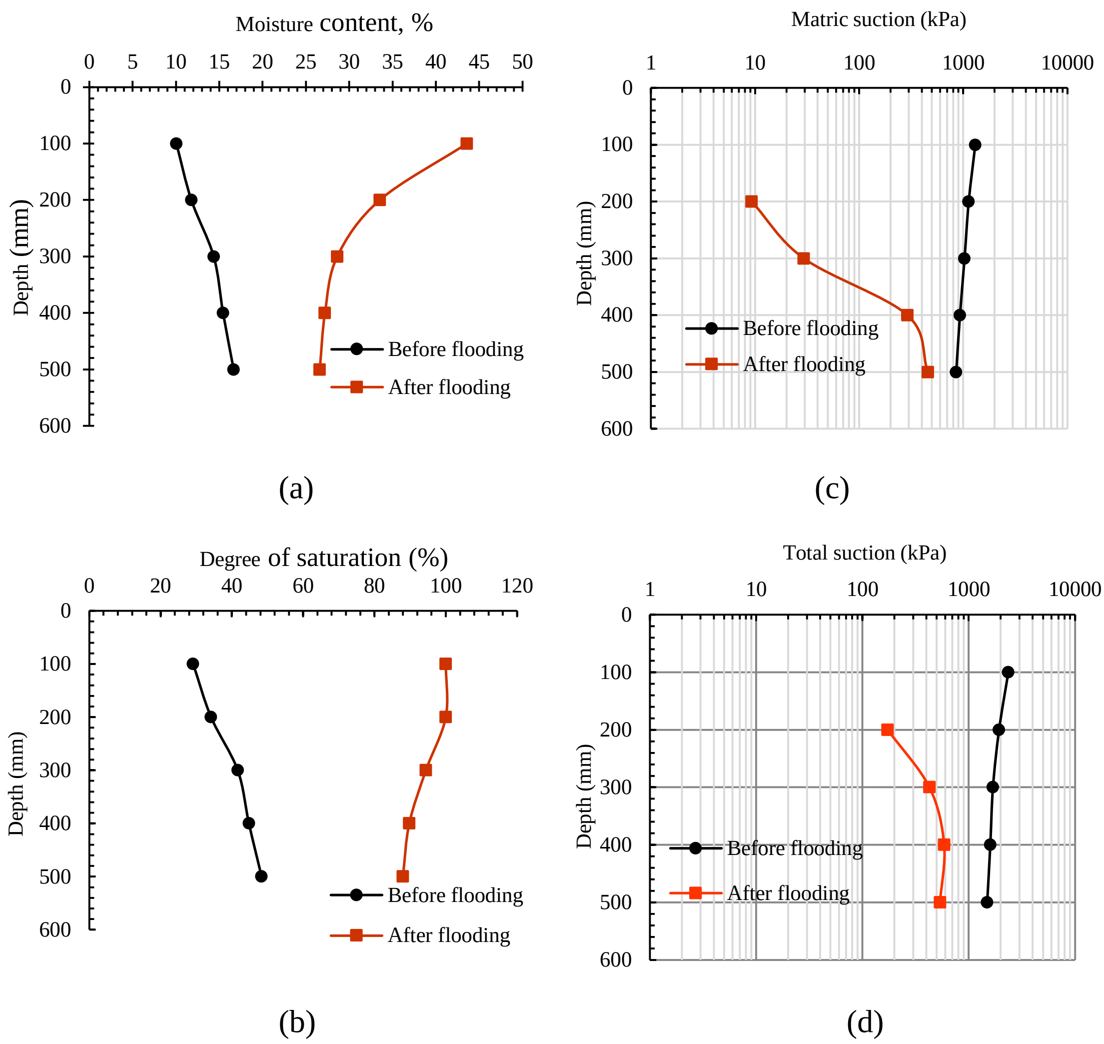


Figure 4.6 Variations of (a) moisture content, (b) degree of saturation, (c) matric suction and (d) total suction after flooding at different depth under the middle sleeper

4.4.1 Track performance after flooding

After half million cycles were applied in Phase-I, the track was flooded for a week to investigate the track performance in a wet (saturated) condition. The water was allowed to drain from the tank and immediately placed under the LOS. In reality, trains run on track immediately after water drainage or even run with water inside the track. Figure 4.7 shows the settlement behaviour before and after flooding of the track at the middle sleeper at 1×10^5 cycles. The track settlement increased by a factor of 9. The first stage track settlement was approximately 22mm. The second stage settlement was approximately 60mm after 1×10^5 cycles. At this level, the loading frequency was reduced from 3Hz to 2Hz. During this phase, the subgrade moisture content, void ratio and soil suction was measured under the middle sleeper only. The surface moisture content was above 40%

and the void ratio reduced to 0.80 (from 0.91). The soil suction was not found because the surface layer became fully saturated.

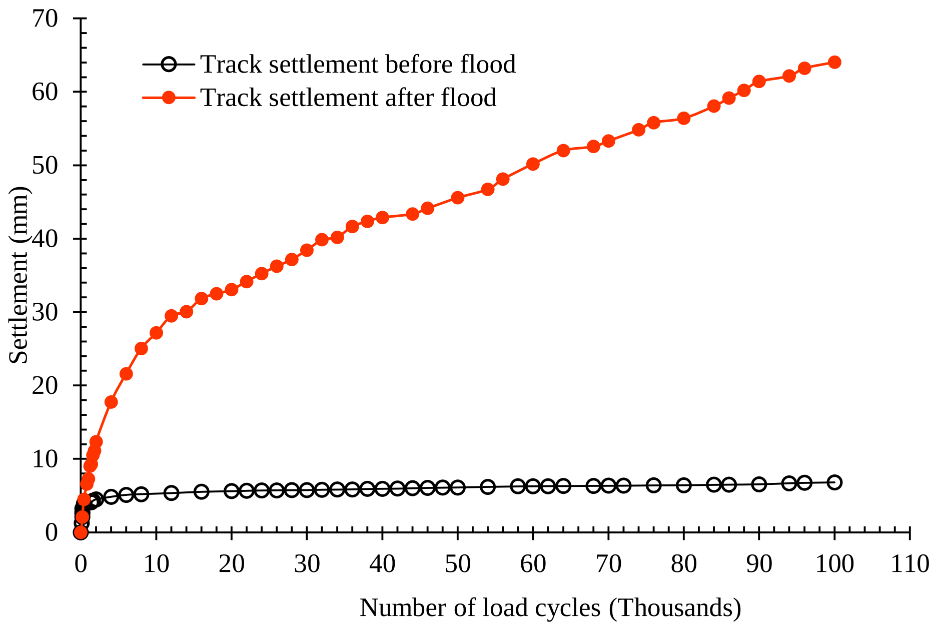


Figure 4.7 Middle sleeper settlement behaviour after flooded track

Figures 4.8 presents the ballast settlement under the middle sleeper before and after flooding. It also showed that a higher ballast movement into the subgrade soil after flooding.

(a)

(b)

Figure 4.8 Ballast settlements (a) before flooding and (b) after flooding under the middle sleeper

The track surface layer became fully saturated and therefore there was no soil suction at that level, as a result, track settlement was high due to loss of suction. Brown (1996) reported that dry soil, which is placed well above the water table, the suction would be high, thus the effective stress will be increased; whereas under wet conditions, the suction will be reduced which will reduce the effective stress.

The two main functions of the subgrade soil are (i) to bear the traffic load without damage and (ii) to drain off the water to the sides of the track. If the bearing capacity is low, then the drainage could fail due to the development of water pockets. Consequently, the soil becomes weak due to the presence of water, therefore the necessary stability may not be maintained which can lead to subgrade failure (Wenty, 2005). Li and Chrismer (2009) reported that flooding limited the ability of the track and the subgrade deformed rapidly due to an increase of subgrade stress.

Ionescu (2004) also observed sudden ballast settlement after flooding with a 40% increase in settlement due to flooding compared with the total settlement in the dry condition. Rapid track settlement due soft track bed occurred because of ballast penetration into the soft subgrade soil; excessive settlement could create pockets in the subgrade which can collect water, causing further weakening of the subgrade as a result of ballast punching into the underlying soil (Burrow et al., 2007). Li and Selig (1998) also stated that the water entering from the ballast layer forms water pockets which trap the water thereby inducing subgrade soil failure. Progressive shear failure develops at the surface layer due to repeated loading.

4.4.2 Track behaviour after tamping (after flooding)

Maintenance was necessary after a 1×10^5 cycles; which entailed removal of the I-section beam in order to tamp the ballast. After tamping, a further 50,000 cycles were applied at 2Hz where, the settlement was rapid (2mm/1000cycles). Figure 4.9a shows the track performance in the wet (saturated) condition after tamping maintenance however, tamping did not improve the track performance. Each time both the primary and secondary settlements were higher resulting from the soft track bed which was unable give support to the track.

The track settlement was approximately 30mm after the second tamp and only 30,000 cycles of loading had been applied is shown in Figure 4.9b. The first and second phase settlement occurred rapidly which can be explained as the ballast had penetrated into the soft soil. The test was stopped to bring the track back up to level and thereafter another 30,000 cycles were applied, resulting in a settlement of approximately 40mm (Figure

4.9c). A fourth tamp was undertaken where settlement was above 30mm after 20,000 cycles were applied (Figure 4.9d).

An important aspect of the work shows, in poor subgrade soil tamping does not work well; moreover, it damages the ballast's properties. At the end of the test, small particles were observed during the ballast removal. Wenty (2005) reported that frequent tamping and very quick return of ballast fouling after track undercutting and ballast cleaning indicates subgrade failure; another disadvantage of tamping is that it loosens the ballast and also fractures the particles (Selig and Waters, 1994; Esveld, 2001). The ballast is highly dependent on the performance of the subgrade. It was shown the ballast tamping does not correct poor subgrade; moreover, it could damage the ballast and may not be economical (Selig and Waters, 1994; Selig and Cantrell, 2001; Audley and Andrews, 2013).

Selig and Waters (1994) reported that the ballast causes both average and differential settlement between surfacing operations which is known as short-term settlement, whereas long-term settlement is related to the subgrade soil; therefore, after flooding tamping was unable to bring the track back up to level due to the poor subgrade.

The track was allowed to dry out to investigate how many days it would take to regain strength without further maintenance.

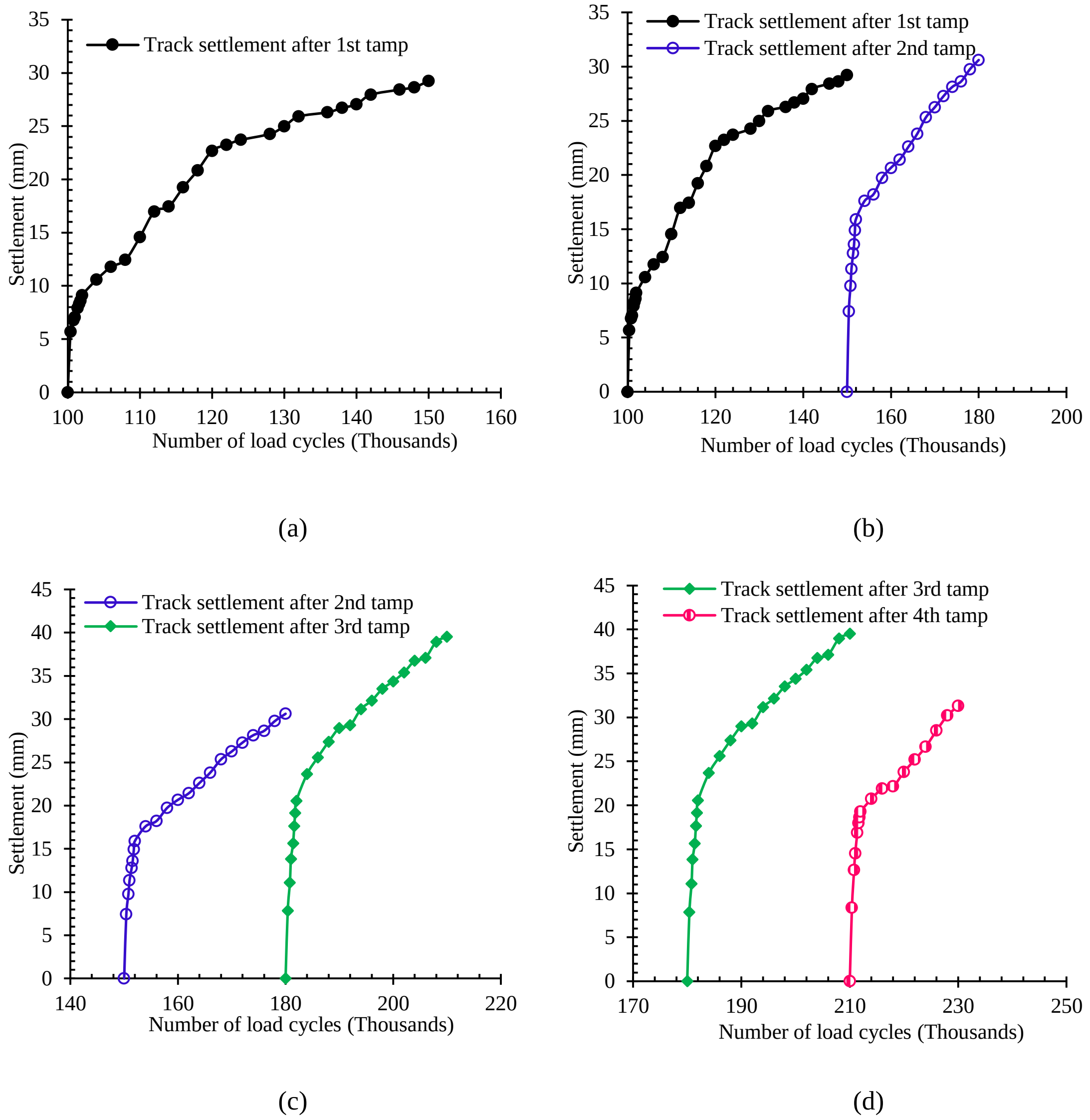


Figure 4.9 Middle sleeper settlements after (a) 1st tamping, (b) 2nd tamping, (c) 3rd tamping and (d) 4th tamping

4.5 Track performance during drying period at Phase-III

Two days later (i.e. after the end of the 4th tamp), soil samples were collected with no load applied on the track. The samples were collected over the 400mm under the middle sleeper to measure the soil properties such as moisture content, soil suction and void ratio. The surface layer moisture content decreased to approximately 36% and void ratio was 0.80 (Figure 4.10). The surface layer was fully saturated, therefore no soil suction was measured. Nonetheless, at other depths, moisture content increased and soil suction decreased due to downward water movement (equilibrium soil water content).

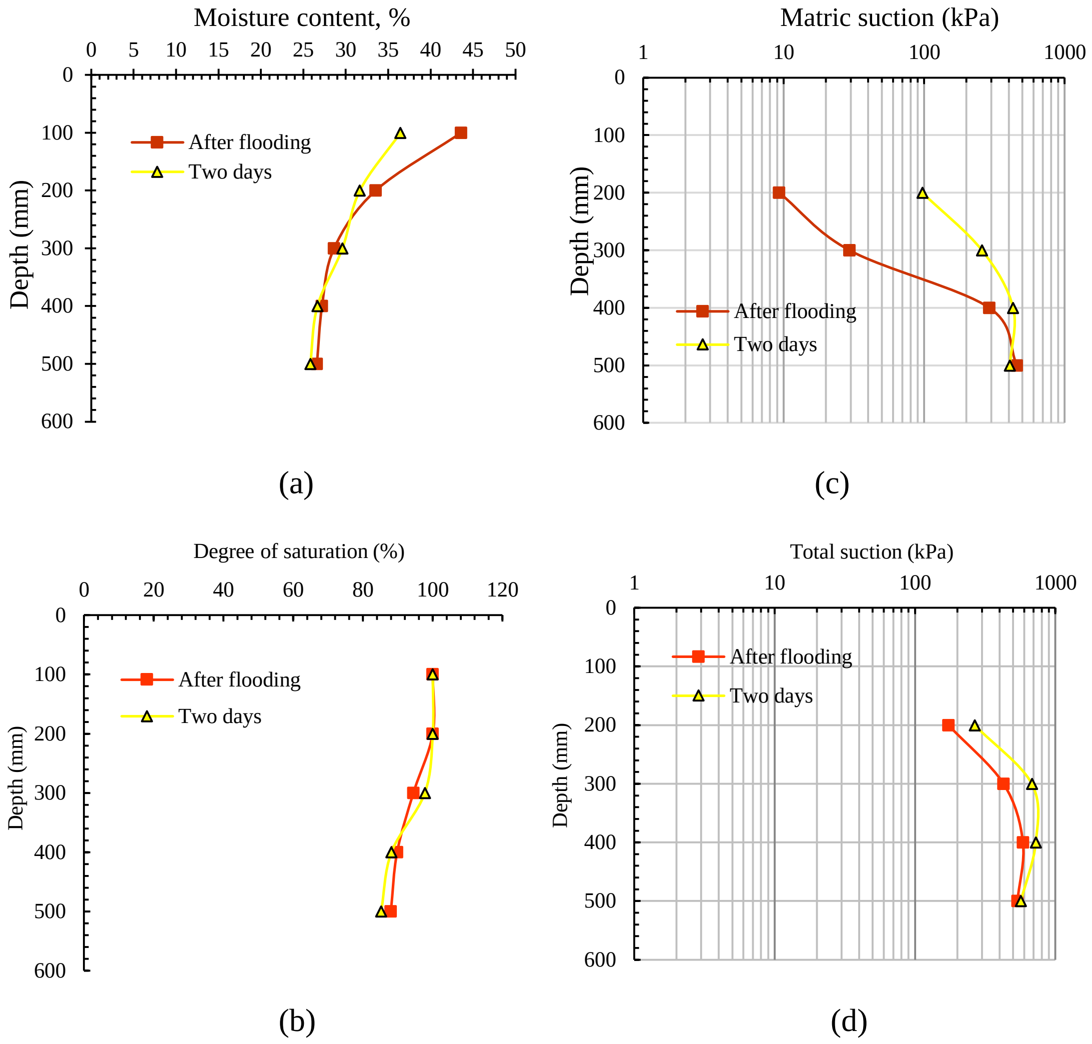


Figure 4.10 Variations of (a) moisture content, (b) degree of saturation, (c) matric suction and (d) total suction after two days at different depths under the middle sleeper

Water movement into the soil is complicated, particularly, under cyclic loading. In transportation systems, cyclic loading causes differential settlement which causes safety problems and also increases maintenance costs. A small change of water content or degree of saturation will have a significant impact on the stress-strain behaviour of soil under cyclic loading (Miller et al., 2000).

One week later the track geometry was again measured, the moisture content remained high at approximately 35.17% (Figure 4.11a) and the degree of saturation was 100% (Figure 4.11b). However, the middle section of subgrade (300-500mm) moisture content was approximately 25%. The soil suction profile was decreased in the subgrade as the entire subgrade was reaching a state of equilibrium state (with compared after two days data) are shown in Figure 4.11 (c) and (d). The surface layer was remained saturated; at this point no load was applied on the track and it was allowed to dry for a further two weeks.

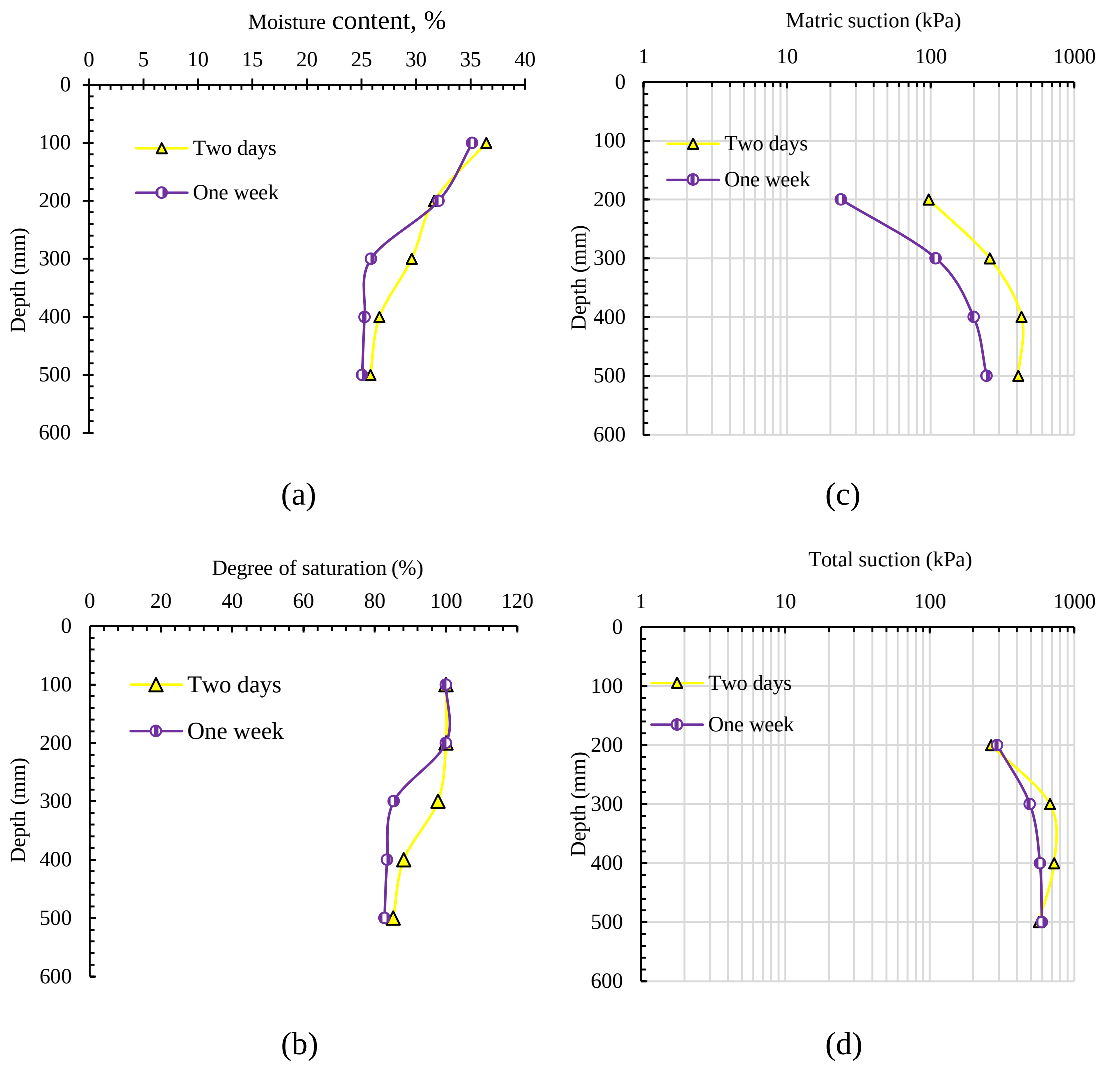


Figure 4.11 Variations of (a) moisture content, (b) degree of saturation, (c) matric suction and (d) total suction after one week at different depth under the middle sleeper

4.5.1 Track settlement after two weeks

The moisture content in the surface layer was approximately 34.63% (Figure 4.12a) and the degree of saturation was 100% (Figure 4.12b). In the other depths, the moisture content and the degree of saturation was not changed significantly. The matric suction (Figure 4.12c) at 500mm decreased by approximately 40%. However, at upper depth, the matric suction increased in comparison with one week data. At the depth of 100mm, some suction value measured (1.74kPa), it indicates the presence of some air between soil particles.

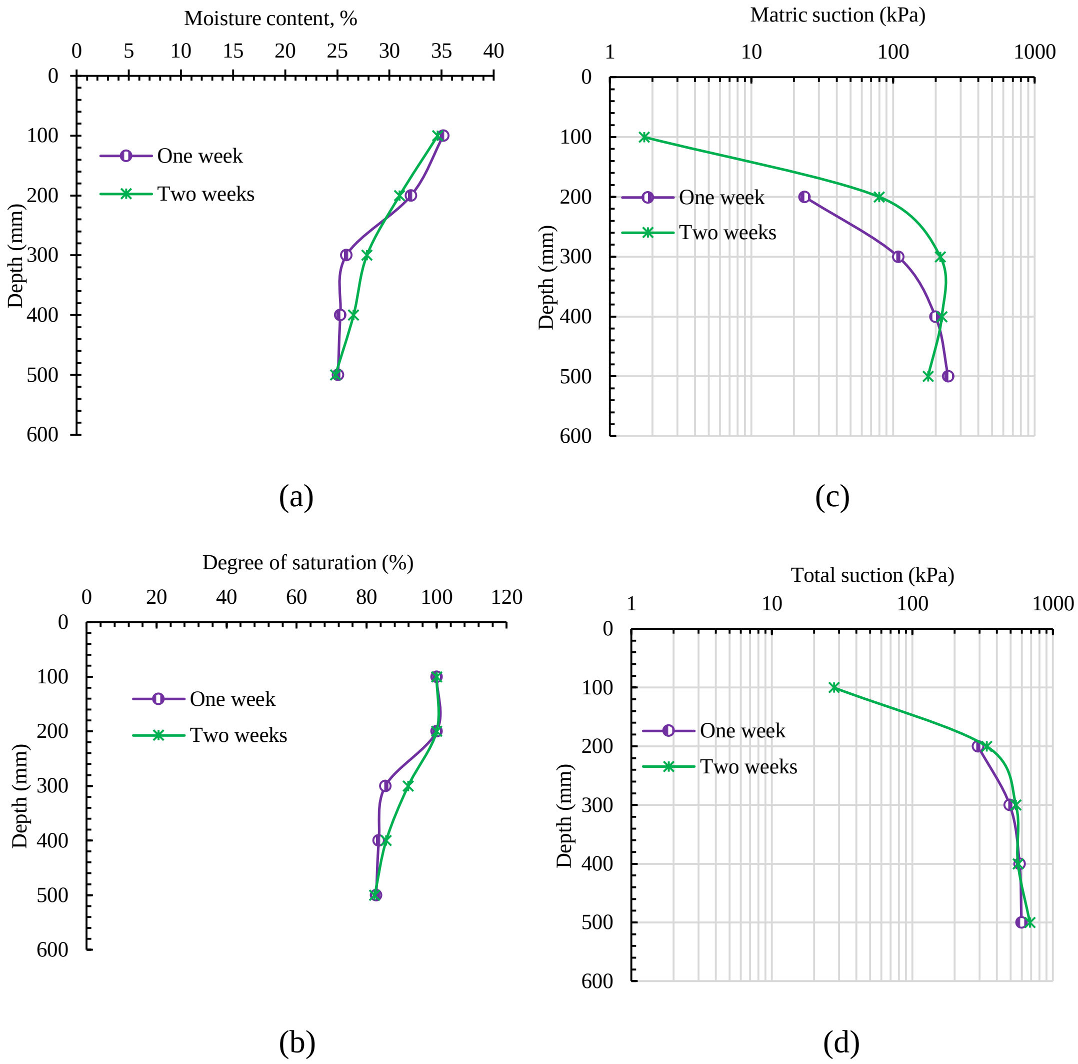


Figure 4.12 Variations of (a) moisture content, (b) degree of saturation, (c) matric suction and (d) total suction after two weeks at different depth under the middle sleeper

The track performance after two weeks (black curve) is presented in Figure 4.15. An important aspect of the research shows, after two weeks of drying the track did not show any improvement. The track settlement was approximately 20% higher than when the test was run immediately after flooding (grey curve in Figure 4.15). The track is not safe to be operational after two weeks even the water drain. After applied 20,000 cycles at 2Hz, the settlement was approximately 38mm. Some water weeping out slowly from two bottom drainage holes during loading. The test was not carried out to avoid further damage to the track. This outcome can be explained by the water movement into the subgrade and the presence of water at the bottom section of the subgrade. The inter-particle menisci bonds become weak due to the loss of soil suction in the subgrade. The subgrade soil behaviour dominated by the bulk water.

4.5.2 Track settlement after four weeks

The track was allowed to dry for a further four weeks to investigate the improvement of the track. The surface moisture content of the subgrade soil was approximately 28% (Figure 4.13a) and the degree of saturation was approximately 91% (Figure 4.13b). Although, the bottom section (500mm) of subgrade moisture content (approximately 30%) decreased by 18% in comparison with two weeks data. The surface layer of subgrade matric and total suction was approximately 100kPa and 250kPa respectively. The bottom section (500mm) of subgrade matric suction was decreased by approximately 38% (Figure 4.13c) and the total suction also decreased by approximately 40%. The moisture content decreased and the soil suction increased at the upper section and the moisture increased and the soil suction decreased at the bottom section (Figure 4.13).

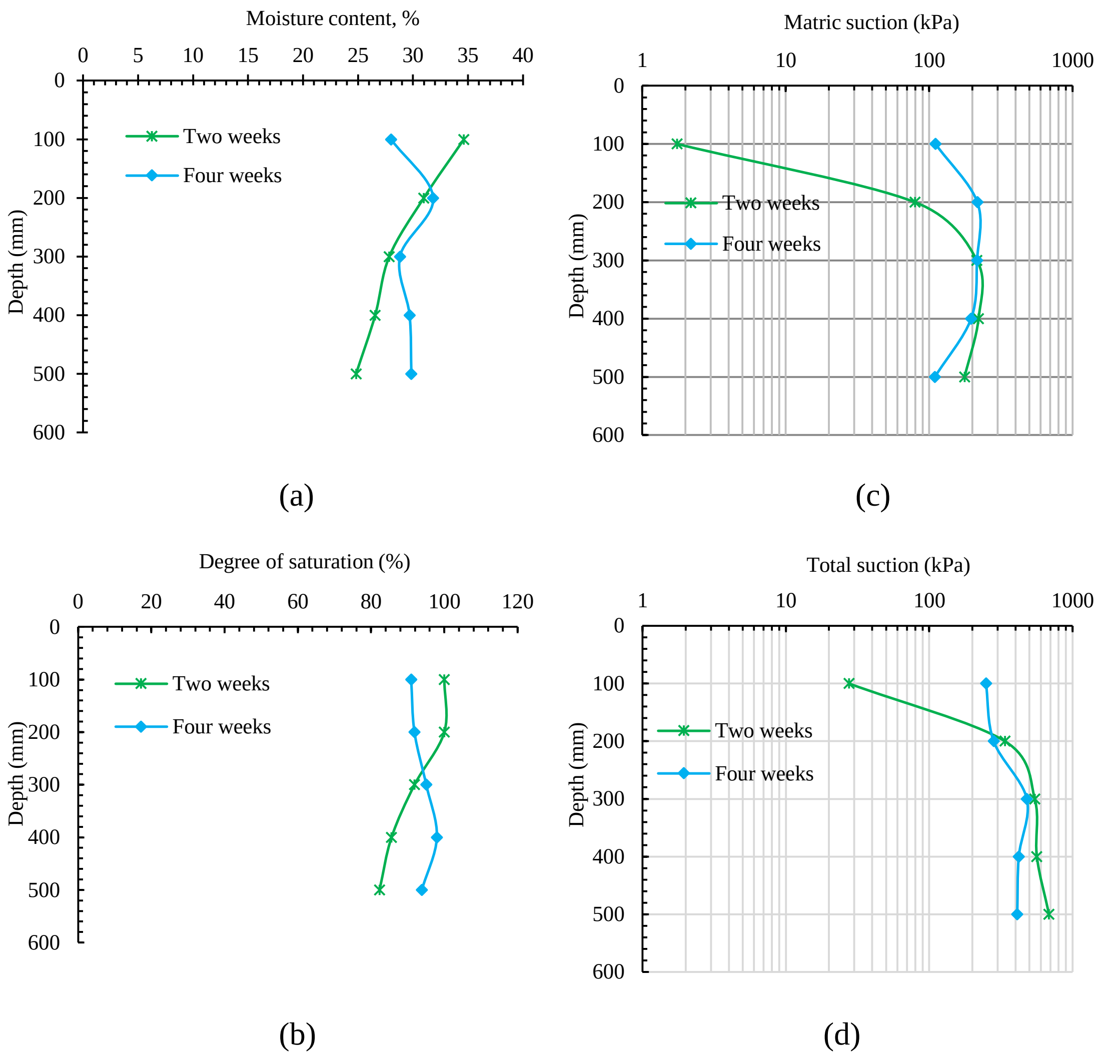
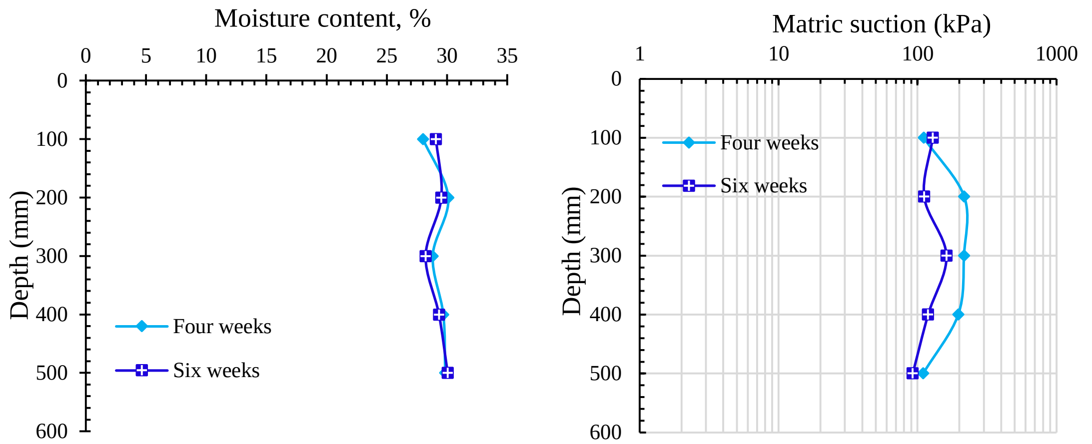


Figure 4.13 Variations of (a) moisture content, (b) degree of saturation, (c) matric suction and (d) total suction after four at different depth under the middle sleeper

The first stage settlement was approximately 15mm and at the end of test, the settlement was approximately 35mm after only 20,000 cycles at 2Hz; this is shown in Figure 4.15 (blue curve). The settlement was decreased by approximately 30 % and 9% in comparison with the test immediately after flooding and the previous stage (after two weeks) respectively. Some water was weeping out during loading from the two bottom drainage holes.

4.5.3 Track settlement after six weeks

The track was placed under the LOS after six weeks. It was observed that the track performance did not improve remarkably. The entire subgrade moisture content and the degree of saturation varied between 25-30% (Figure 4.14a) and 82-99% (Figure 4.14b) respectively. The matric suction varied between 100-250kPa (Figure 4.14c) and the total suction 400-460kPa (Figure 4.14d). The soil suction (matric and total) of subgrade lower section (500mm) decreased and moisture content increased in comparison with four weeks data. The soil suction reaching in an equilibrium state is shown in Figure 4.14 (c) and (d).



(a) (c)

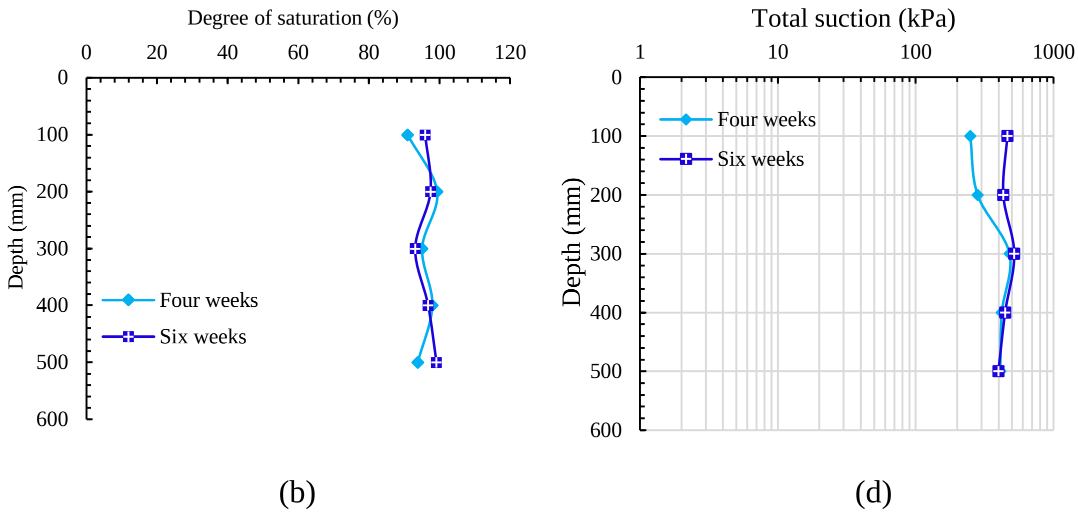


Figure 4.14 Variations of (a) moisture content, (b) degree of saturation, (c) matric suction and (d) total suction after six weeks at different depth under the middle sleeper

The settlement was found to be 30mm after 20,000 cycles at 2Hz (pink curve in Figure 4.15). By comparison, with the test immediately after flooding, after two weeks and four weeks, the settlement decreased by approximately 18%, 26% and 8% at the drying period respectively. Although, the track was yet showing poor performance, subgrade soil could not give sufficient support; on the other hand, track bed was covered by ballast, therefore, track was drying slowly.

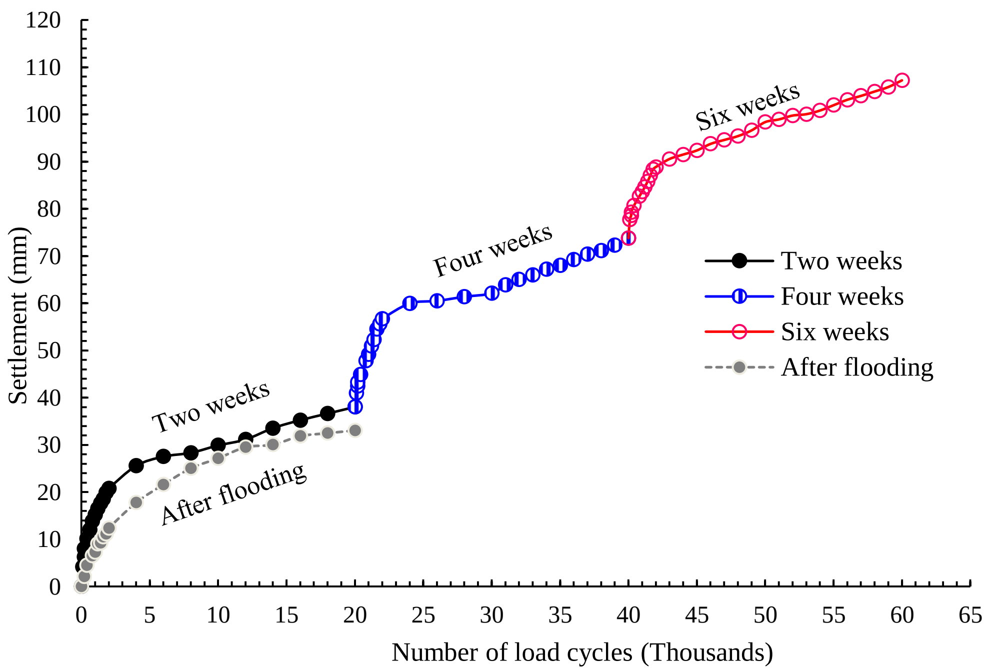


Figure 4.15 Middle sleeper settlement after two weeks, four weeks and six weeks

During Phase-III, the track performance improved at the end of six weeks but during the first few weeks, the worst track performance was noticed due to water movement. Another problem that was found was the tilting of the track on the drainage side. The track tilted to drainage side where some water was found after removing the ballast is shown in Figure 4.16. Also at this side, subgrade was softer compared to the subgrade on the non-drainage side.

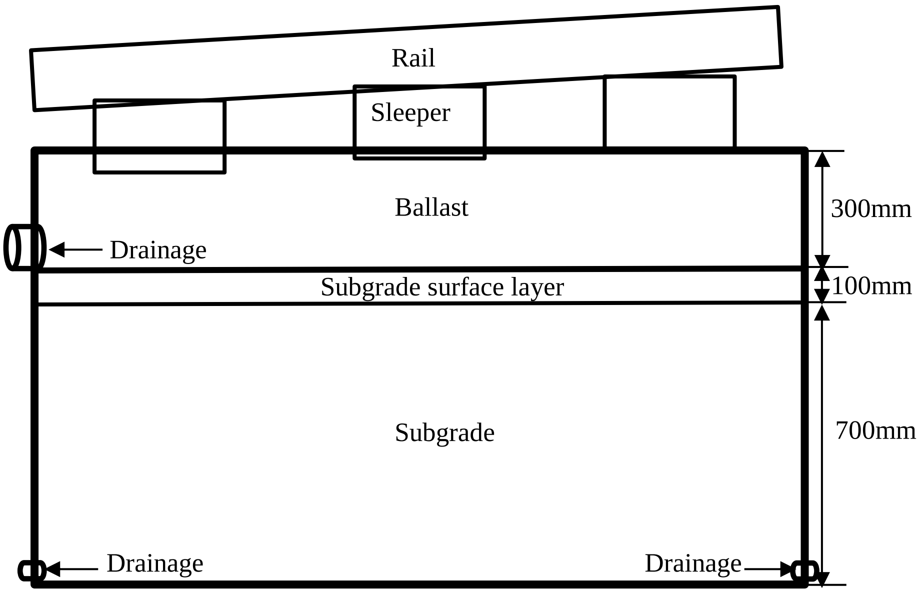


Figure 4.16 Track tilted during loading towards drainage side

4.6 Subgrade problems after flooding

Significant softening occurred due to flooding. In the end of the Experiment-1, on removal of ballast from the tank, visual inspection found extensive ballast penetration into the formation layer as shown in Figure 4.17. The combination of water and cyclic loading served to produce a slurry. Ayres (1986) documented that the mud pumping is formed in two ways (note-the second cause is not related to this research): (i) erosion pumping failure where ballast particles penetrate into the subgrade thereby forcing soil particles upwards and (ii) dirty ballast pumping failure which happens due to wind-blown deposits, brake dust and dirt dropping from the train. Slurry could substantially reduce ballast modulus of elasticity (Sharpe and Caddick, 2006). This is one of the reasons of poor track performance after flooding.

Figure 4.18 shows the subgrade soil after removing all the ballast at the end of Experiment-1. Removing the ballast from the formation layer was difficult; because the ballast was held by the soil very firmly. Some of the ballast was severely caked in mud (Figure 19) which required to be replaced. Some crushed ballast particles that were also noticed during ballast removal, perhaps resulting from frequent tamping during the wet phase. The surface was uneven after the ballast was removed and considerable softness of the subgrade was found near the drainage side.

Figure 4.17 Ballast penetrations after flooding at surface layer

Figure 4.18 Track bed after removing the ballast

Figure 4.19 Foul ballast after flooding

4.7 Concluding Remarks

The test results confirmed a significant difference in track performance (in terms of settlement) before and after flooding. Track is not safe to be operational immediately after the drained water from the track, without an investigation of soil properties including moisture content, suction and the degree of saturation. In the event of flooding, the upper layer of the subgrade is most affected and sensitive to changes of water content and soil suction. However, over time, the entire subgrade can be affected if the water stays in the track for a long period. An effective drainage system is required to direct water away from the track as quickly as possible. The research suggests that, without checking the subgrade properties, the track should not be open for operation and train services should be halted.

For the periods of scheduled maintenance work, all the attention is given to ballast properties. However, subgrade properties should be checked as well, especially after an extreme event, for example rainfall sufficiently heavy to result in flooding. The results showed that if tamping of the track does not work, then it is a fault of the subgrade. Therefore, it cannot bring the track up to level due to permanent settlement; in addition, frequent tamping can damage ballast properties.

During the recovery period over the first two weeks, the track showed little improvement. The subgrade soil properties indicated that the subgrade soil had reached an equilibrium condition. Consequently, track settlement increased significantly, particularly after two weeks of the recovery period. However, four and six weeks later the track settlement decreased, but not significantly. The research indicates the wetting process is considerably faster than the drying process. In this situation, extra care was required to protect the track from further damage; for instance, load restriction can be applied.

The performance varied with changes of soil suction; higher suction values specify a stiffer track, whereas lower suction indicates poor performance. The settlement of the track is much higher in the wet condition due to the loss of soil suction. The results also confirmed that the subgrade properties did not change significantly, as the subgrade is covered by the ballast. However, the subgrade soil dried out quickly after removal of the ballast. Finally, this research suggests more attention should be given to subgrade behaviour; particularly checking subgrade properties to avoid further track damage and reduce maintenance costs.

5.1 Introduction

This chapter reports and discusses the influence of traditional sand blanketing on track behaviour. Traditionally, sand blanketing is one of the common techniques that has been used as a drainage material as well as to protect the subgrade from erosion. A sand blanket is generally a permeable layer of fine granular material which is placed as a drainage layer on subgrade soil to allow water to drain from the subgrade surface (Bonnet, 2005). The sand blanket also prevents ballast penetration into the subgrade soil, together with the pumping action of clay (Selig and Waters, 1994; Li et al., 2007b).

Progressive shear failure occurs due to over stressing on the clay subgrade soil; an event which can be avoided by placing granular material to enhance drainage. Wenty (2005) reported attrition results in the development of a slurry of the ballast-subgrade interface due to the presence of water and heavy dynamic loading. Overloading the subgrade creates water pockets which, as a result, cause attrition and can be avoided by placing a granular blanket. Sharp and Caddick (2006) reported that the sand blanket prevents upward movement of the slurry by filling the voids in the subgrade. If slurry is formed under the clay it is retained in the clay, hence it dries out in time, sand blanket therefore increases granular layer stiffness. The track becomes vulnerable in the wet condition; a situation which impacts on each component of the track, particularly the subgrade soil. To protect the subgrade soil, it is important to know that the soil's behaviour, specifically its drainage system. Sand blanketing is a common and useful method of protecting subgrade soil but it cannot overcome all the water related problems. Sometimes, it creates additional problems to the track if the drainage system is not efficient.

Experiement-2 is divided into three phases which are described as below:

Phase-I (after flooding with a sand blanket): After removing all the ballast the soil samples were collected at different depths under the three sleepers to measure the soil properties. The aim of this phase was to investigate the influence of a sand blanket at saturated conditions. The moisture content of the subgrade surface was approximately 27% (see Figure 5.2a). The subgrade matric and total suction was between 150-570kPa respectively (see Figures 5.2c & d). A 150mm sand blanket was placed on the subgrade according to Network Rail standard RT/CE/S/033. The optimum moisture content of sand was 12.8%. The track was flooded for a week as in the first experiment. After this time, the water was then allowed to drain for a week, after which it was placed under the LOS. It was ensured that there was no water coming out from the drainage holes. The test was repeated after two, four and six weeks to record any improvement of the track. Track performance and behaviour are discussed in section 5.3.

Phase-II (with water inside in the GRAFT and a sand blanket): In this Phase, the flooded track was placed under load without the water being drained. After the first Phase was completed, the moisture content increased over the entire subgrade, particularly at the surface and bottom sections. The water passed through at the side wall of the tank. The surface moisture content was above 35%. The surface layer became fully saturated (degree of saturation = 100%). The matric suction was between approximately 50-80kPa at the depth between 200-400mm. The total suction varied between 100-300kPa (at the depth between 200-400mm). The test was repeated after four and six weeks. A discussion about the track performance during this phase is presented **section 5.4**.

Phase-III (Dry phase with a new surface layer): The surface layer of the track became fully saturated and the test was discontinued to allow further maintenance. The subgrade surface layer (100mm) was replaced by a new surface layer. This Phase investigated the track performance in dry condition. At the bottom depth, (500-700mm) the subgrade soil was almost saturated. The moisture content was approximately 28% and the degree of saturation was 98%. The track often experiences seepage problems in real life in the wet period or in wet conditions. The water can get into the track from the bottom layer of soil if the water table rises (Figure 1.1). The track behaviour of this phase is discussed in section 5.5.

5.2 Influence of sand blanketing on track performance

Sand blanketing is a traditional technique to improve drainage facility. Figure 5.1 shows a layer of sand blanket placed on subgrade. Two tests were conducted to investigate the influence of sand blanketing; both test are described below.

Figure 5.1 A 150mm sand blanket layer placed on subgrade soil

5.3. Track performance after flooding at Phase-I

The moisture content of the subgrade surface layer (100mm) was approximately 27% and the degree of saturation was 90% are shown in Figures 5.2 (a) and (b). The matric and total suction was measured approximately 150kPa (Figure 5.2c) and 560kPa (Figure 5.2d) respectively. In the other depths (up to 500mm), the moisture content averaged 25% and the degree of saturation averaged 80%. However, at 300mm depth, the moisture content was found to be approximately 21% and the degree of saturation was approximately 70%. The matric and total suction was measured at approximately 540kPa and 720kPa respectively.

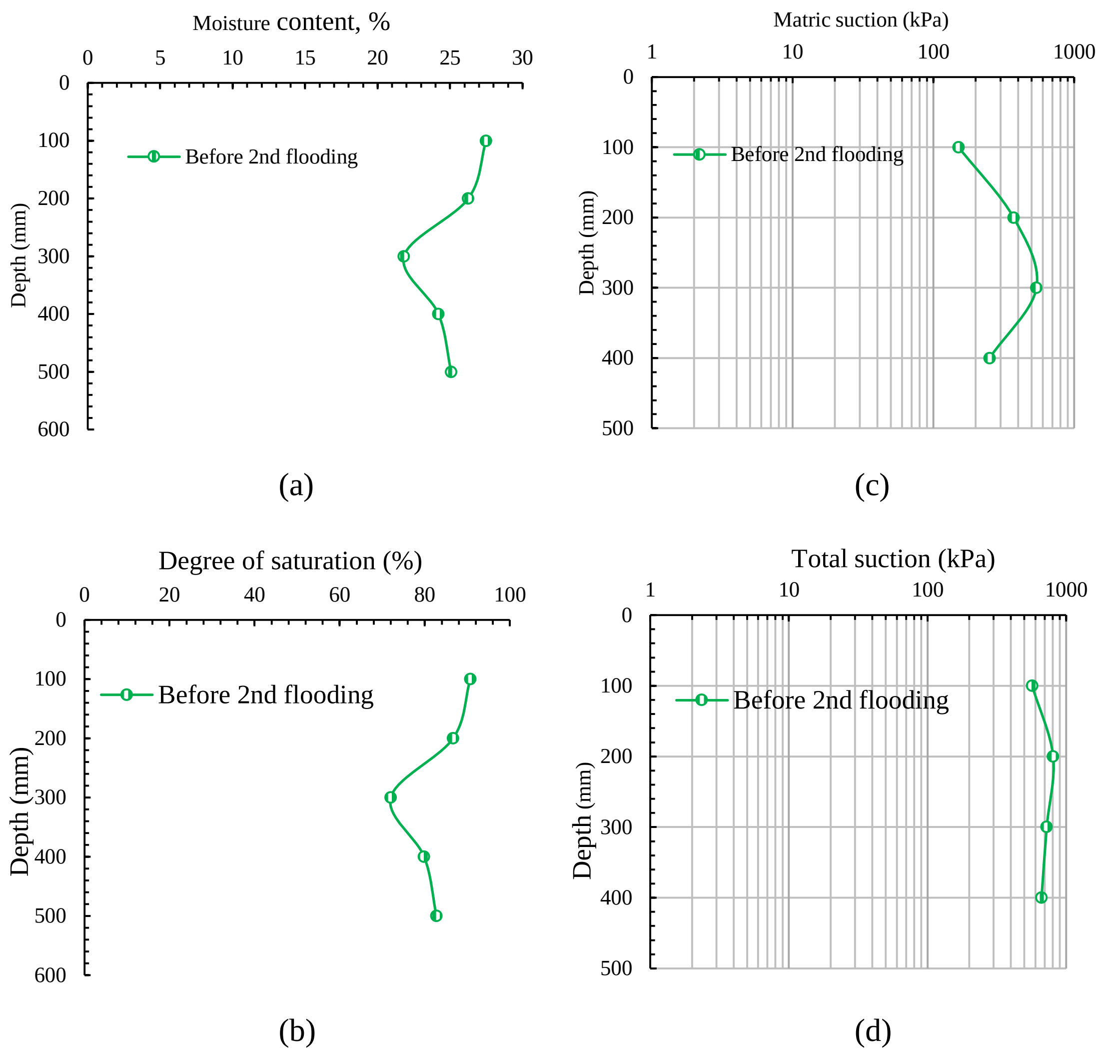


Figure 5.2 Variations of moisture content, matric suction and total suction in Phase-I under middle sleeper

In this phase, the track was flooded for a week with a 150mm sand blanket placed on the subgrade soil, water was then allowed to drain from the track. Figure 5.3 shows the track performance after placement under the LOS. The track's first stage settlement was rapid and was approximately 25mm due to the wet surface of sand blanket. After only 1400 cycles, the track settlement was approximately 45mm, which is 5 times higher than the track without sand blanketing at the same loading cycles (see Figure 4.7). This outcome can be explained, the sand blanket layer liquefied due to the flooding regime. On the other hand, fine particles migrated into the ballast which induced rapid track settlement. In addition, the track already experienced flooding which changed the subgrade's properties as shown in Figure 5.2. The entire subgrade moisture content increased and soil suction decreased due to repeated flooding. The cyclic loading started at 2Hz; after a while it was noticed that, the rail was tilted to the drainage side (see Figure 4.16) and, as a result, the cyclic loading was reduced to 1Hz. The test was stopped after 1400 cycles due to the rapid settlement of the track.

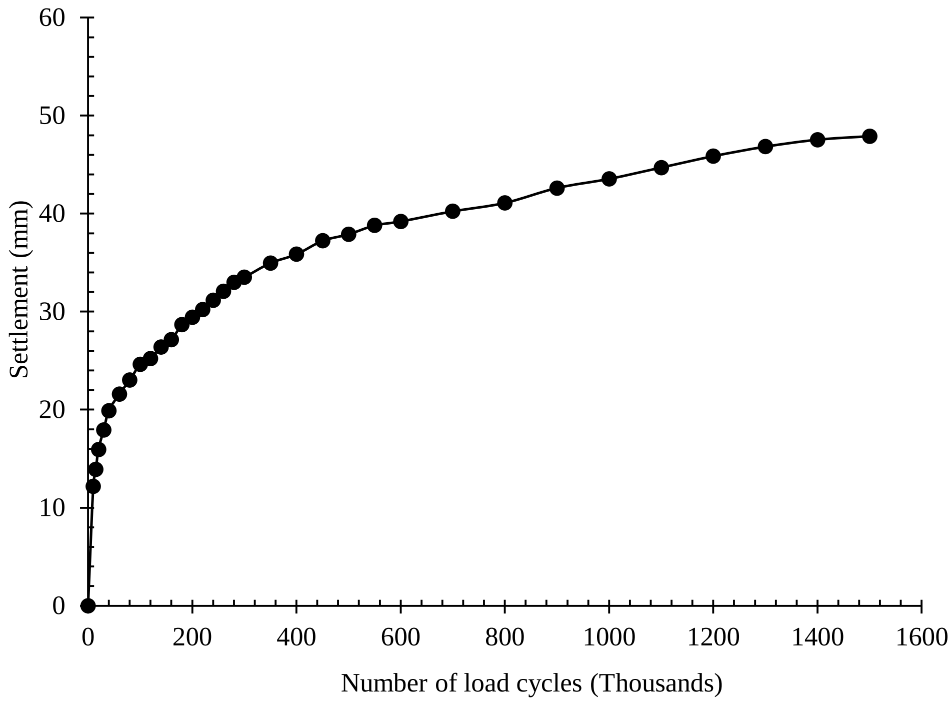


Figure 5.3 Middle sleeper settlement behaviour after flooded track with sand blanketing

The traditional sand blanketing technique is widely accepted as a means of enhancing surface drainage and protecting the subgrade from erosion related problems. However, it is difficult to maintain for the engineers attempting to keep the track operational due to the large volume of excavation and the need for the importation of new materials (Sharpe and Caddick, 2006). On the other hand, a sand blanket is unable to give adequate support to track, if it becomes fully saturated. If that happens it causes serious subgrade problems including upwards migration of fines, ballast fouling and slurry formation are likely to be caused. (Li et al., 2007b) reported from their cyclic loading test (with a sand blanket) that the cyclic loading (passing train) can cause hydraulic pumping of slurry into the ballast layer, thereby rising progressively up to the ballast surface. The slurry lubricates ballast particles, inducing track settlement, which is accompanied by the loss of thickness in the subgrade soil.

After removing the surface ballast, fine particles were seen on the ballast is shown in Figure 5.4. Figure 5.5 (a), (b) and (c) show the clogged ballast with fine particles, with a layer of ballast having been penetrated into the sand blanket. Mud pumping is a major problem of the track bed which occurs due to a combination of fine particles and water. Ayres (1986) stated that the track performance could be poor due to slurried ballast despite a high strength subgrade.

Figure 5.4 Fine particles movement in to the ballast

(a)

(b)

(c)

Figure 5.5 Ballast in the sand blanket after the test (a) ballast penetration into the sand blanket, (b) clogged ballast after flooding and (c) a layer of ballast extracted from the sand blanket

Figure 5.6 shows a noticeable settlement of the subgrade, which was observed after removing the sand blanketing; particularly under the three sleepers. Sharpe and Caddick (2006) reported that sand blankets failed to protect subgrade from erosion on several occasions.

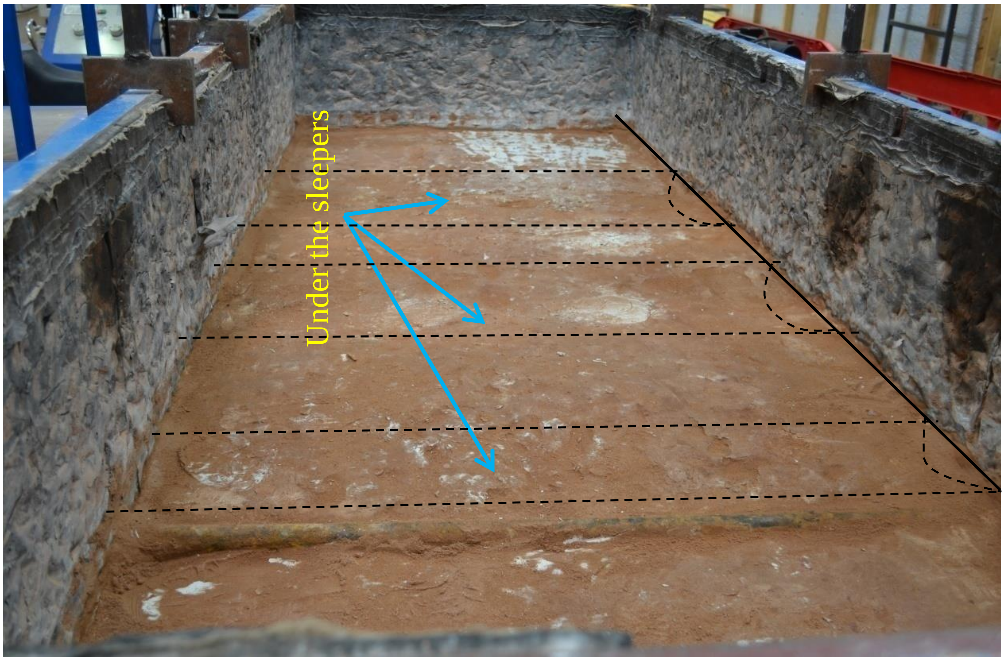


Figure 5.6 Track bed after removing all the ballast and sand blanket

5.3.1 Track performance after two weeks

Track performance did not show any improvement after two weeks. Figure 5.7 presents the settlement behaviour of the middle sleeper after two weeks of drying (black curve). The first stage track settlement was approximately 20mm. After 2000 cycles, the track settlement was approximately 40mm. Track settlement after two weeks without sand blanketing was almost 50% lower than the track settlement with sand blanketing. Some water became entrapped between the subgrade and the sand blanket (see Figure 5.14) which was trickled from the bottom drainage holes during the loading period. The sand layer and subgrade soil with ballast created a ballast pocket; furthermore, fine particles migrated into the ballast which caused excessive settlement.

5.3.2 Track performance after four weeks

After four weeks, a slight improvement was observed shown by the blue curve in Figure 5.7. The settlement decreased by almost 30% compare to two weeks settlement. The first stage settlement was approximately 25mm and after 10,000 cycles, the settlement was approximately 40mm. In comparison with Experiment-1, after four weeks, the track settlement was approximately 60% higher than the track without implying that sand blanketing as presented in Figure 4.15 (blue curve).

5.3.3 Track performance after six weeks

Six weeks later the settlement was almost similar compared to the previous stage (four weeks). Although, the track settlement was almost 30% higher than the track settlement in Experiment-1 without the sand blanket (Figure 4.15) after the same period. The settlement was approximately 42mm after 10,000cycles.

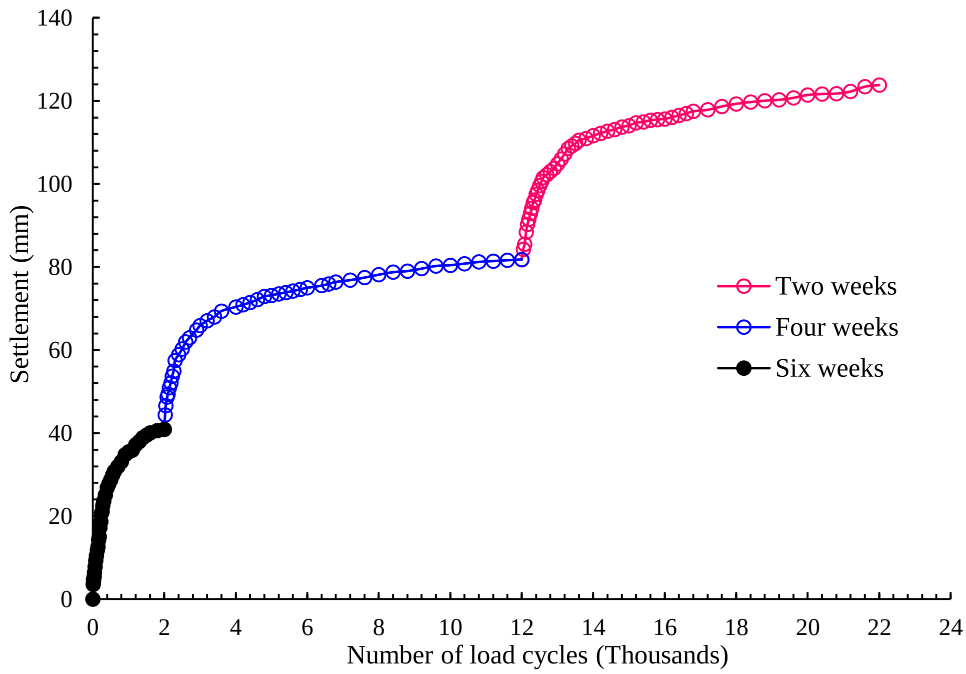


Figure 5.7 Track settlement after two, four and six weeks at Phase –I

5.4 Track performance during the track under water at Phase-II

The track was prepared with a new 150mm sand blanket for the next phase test, where the track was flooded for a week and then, without draining the water, the track was placed under cyclic loading. The ballast was marked to investigate any changes during and after loading; the prepared track is shown in Figure 5.8.

Figure 5.8 Prepared track with sand blanketing before flooding

Subgrade soil properties were measured before and after the test to investigate the subgrade soil behaviour. Before flooding, the moisture content of subgrade soil varied at different depths from between 30-38%. The matric suction was varied between 50-80kPa and the total suction was between 100-300kPa. Compared to the first experiment (without sand blanketing), the track performance showed better than with sand blanketing at the saturated condition. After completing the test, soil samples were collected to measure the moisture content, soil suction and void ratio. The moisture content of the surface layer was above 35% and overall soil suction of the entire subgrade soil had been decreased; within the 200mm of subgrade, soil suction was not found. The void ratio was to 0.79. However, the middle section of subgrade moisture content (22%) was found to be less than the surface layer (100mm) and bottom section (500mm). With every occasion of flooding, the upper section and bottom section experienced flood water affected directly, whereas, middle section increasing moisture content was a capillary effect. A detailed summary of subgrade soil properties is present in Table B.1 in Appendix-B.

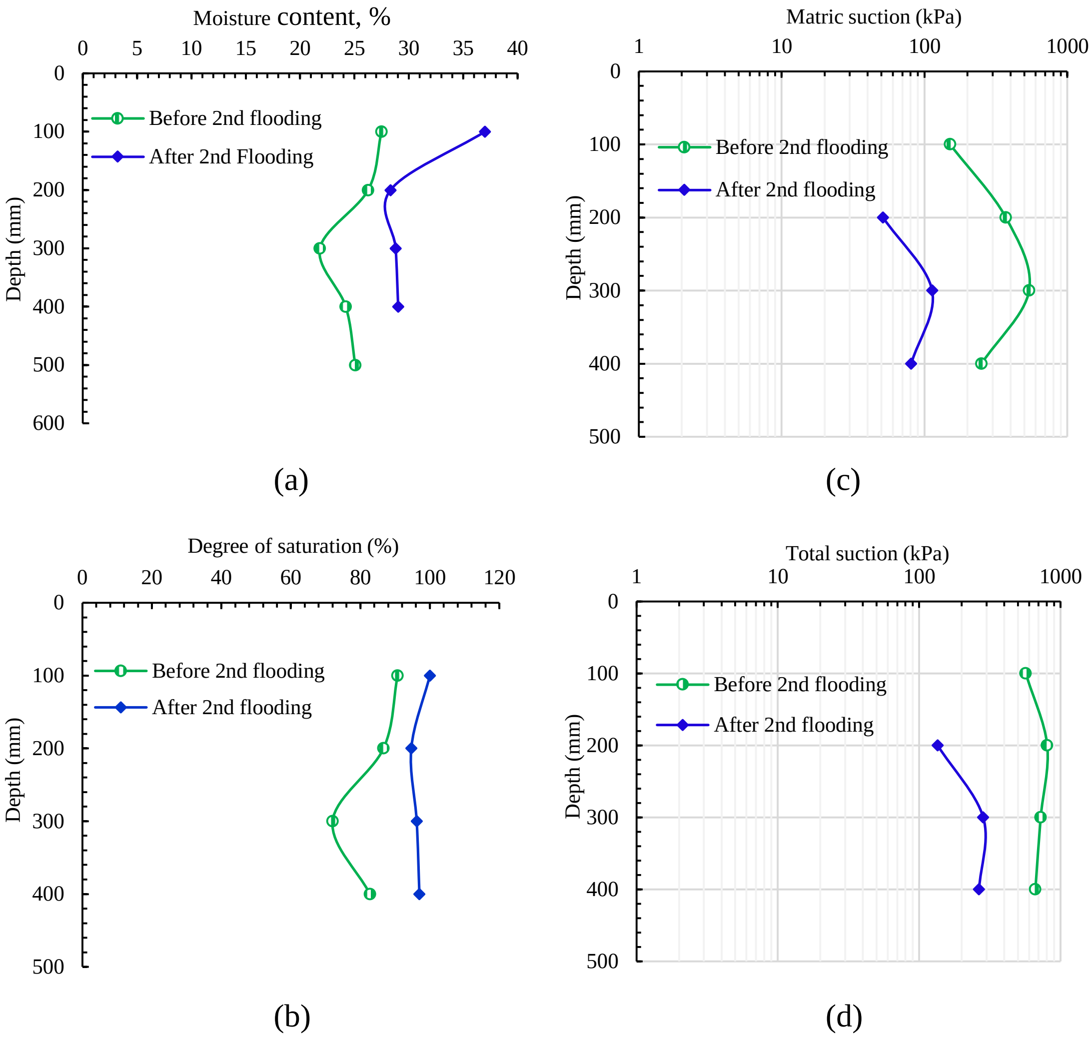


Figure 5.9 Variations of moisture content, the degree of saturation, matric and total suction before and after 2nd flooding

After a week of flooding, the track was placed under loading without drianed water from the track, the drainage was sealed. Figures 5.10 (a), (b) and (c) show the track under LOS before and after loading, and after the test. The track rapidly submerged, only after 1500cycles (see Figure 5.10b). Repeated flooding reduced the subgrade stiffness, M_r = 29MPa (see Chapter 6) and the surface layer became soft. Without undertaking further maintenace the track was not suitable to take any load.

Figure 5.11 shows the track settlement during loading with water inside the track. The first stage settlement was approximately 20mm after only 50 applied cycles. The presence of water in the track resulted in an extreme track settlement of approximately 65mm after only 1500 cycles. Loading was applied at 1 Hz.

(a)

(b)

(c)

Figure 5.10 (a) Flooded track before loading (b) Flooded track after loading (c) Track after the test

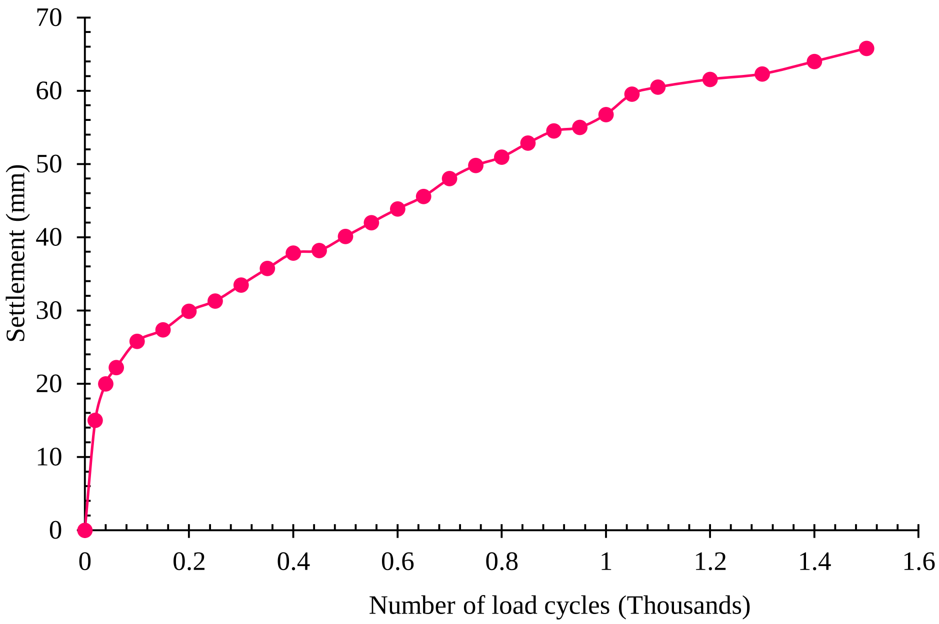


Figure 5.11 Track settlements during track under flood

5.4.1 Track behaviour after four weeks

After draining the water, the track was allowed to dry for four weeks and then placed under loading. The track settlement was almost 50% higher than the previous phase (Experiment-2, Phase-I). The combination of the presence of water and applied cyclic loading, which caused fine particles to move upwards and ballast to penetrate into the soft soil. The track settlement was approximately 60mm after only 2000 cycles. In Figure 5.12, the green curve presents the track performance after four weeks. The load was applied at 2Hz; some water was weeping out during loading.

5.4.2 Track behaviour after six weeks

After six weeks, the track was placed under the LOS and 10,000 cycles applied at 2Hz. The track settlement was approximately 48mm (Figure 5.12) which was 20% higher than the previous stage (Experiment-2, Phase-I). The subgrade became weak, due to repeated flooding, which resulted in it being unable to give adequate support to the structure.

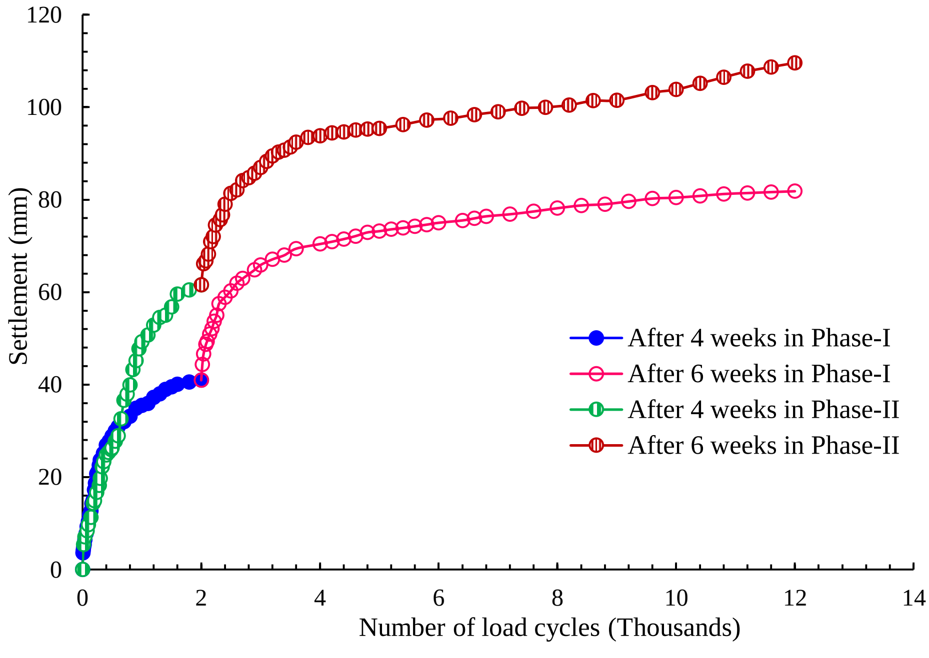


Figure 5.12 Comparison track settlement between Phase-I and Phase-II at four and six weeks

Table 5.1 compares the track settlement between Experiment-1 and 2. The results showed track settlement was lower, during the recovery period without sand blanketing. The blocked drainage or improper drainage facility caused more track problems including ballast fouling, entrapped water and higher track settlement. Figures 5.13-15 show the additional problems that occurred due to blocked drainage.

Table 5.1 Comparison of settlements during recovery period

Experiments	Settlement (mm)	2 weeks (mm)	4 weeks (mm)	6 weeks (mm)
Exp-1, Phase-III	First stage settlement	20	18	15
	Second stage settlement	18	17	18
	Cycles (thousands)	20	20	20
	Reduction in settlement (from 2 weeks)	----	6%	12%
Exp-2, Phase-I	First stage settlement	22	20	20
	Second stage settlement	18	20	22
	Cycles (thousands)	2	10	10
	Reduction in settlement (from 2 weeks)	----	28%	---
Exp-2, Phase-II	First stage settlement	----	15	20
	Second stage settlement	----	46	21
	Cycles (thousands)	----	2	10
	Reduction in settlement (from 4 weeks)	----	---	52%

Figure 5.13 Fine sand migration in to the ballast

Figure 5.14 Water entrapped between sand blanket, ballast and subgrade created ponding of water

Figure 5.15 Muddy and saturated surface layer

After completing the test, soil samples were collected to a depth over 600mm to evaluate moisture content, soil suction and void ratio. At the depths 100-600mm, the moisture content varied between 30-35% and the matric suction varied between from 40-200kPa;

the void ratio was 0.79. Table B.2 in Appendix B presents a detailed summary of the subgrade soil properties at different depths.

5.5 Track performance in dry condition at Phase-III

The surface layer (100mm) became soft and was removed and replaced with a new soil layer (see Figure 5.16). Soil compaction on the wet surface was extremely challenging. On the other hand, in addition the bottom section (500-700mm) of soil became softer (the moisture content = 30% and the degree of saturation = 99%) than the middle section because the water travelled all the way to the bottom of the subgrade through the side wall of the tank on every occasion of flooding. At the beginning, the soil was placed with 12% moisture content and it was compacted both by an electric compactor (40kPa) and manually in three layers; afterwards the tank was placed under the LOS to compact the soil even further.

Figure 5.16 Subgrade soil with new surface layer

After compaction, soil samples were collected from the surface layer to measure the soil properties. The moisture content was approximately 27% and the degree of saturation was 88% (Table B.3, Appendix-B). The matric suction was approximately 100kPa and total suction was approximately 350kPa (Table B.3, Appendix-B). The subgrade was allowed to air-dry to obtain the moisture content of 12%. Prior to starting the test, soil samples were collected (after 8 weeks) at different depths. The moisture content and the degree of saturation at the surface layer were approximately 12% and 40% respectively (Figures 5.17a and b). At other depths, the moisture content and the degree of saturation were approximately 25% and 82% respectively. Between 100-700mm the matric suction was between approximately 200-700kPa and the total suction was between 500-1400kPa (see Figures 5.17c and d). Table B.3, Appendix-B, offers a summary of soil properties at different depths.

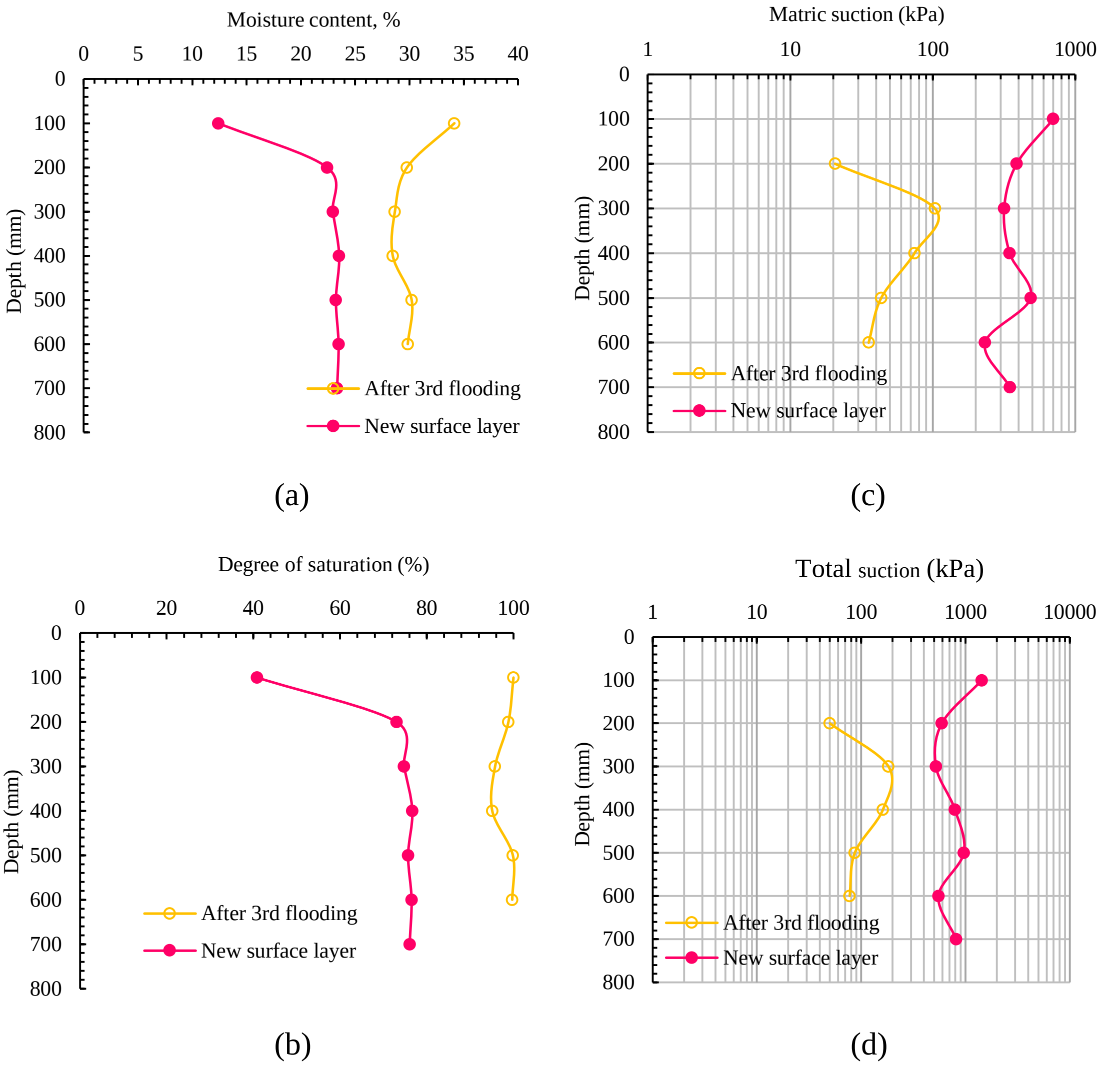


Figure 5.17 Variations of moisture content, the degree of saturation, matric and total suction after 3rd flooding and with new surface layer

The purpose of this experiment was to investigate the effect of a rise in the water-table. During a wet period, a rise in water-table will change the subgrade properties including higher moisture content and lower soil suction. Changes in subgrade soil moisture and hence, in its subgrade modulus can occur over the service life of a pavement system irrespective of the initial moisture conditions imposed during construction. It has been shown that the subgrade modulus varies with moisture content and suction (Sawangsuriya et al., 2009).

Figure 5.18 shows the track behaviour for the new surface layer. The first stage settlement was approximately 15mm after only 2000 cycles; the settlement then increased gradually up to 30mm after 50,000 cycles. It was observed that after 1×105 cycles the settlement rate reduced (0.50mm/10,000cycles) and became linear. After 3×105 cycles no significant changes were observed and so the experiment was stopped at this point. The settlement was found to be approximately 47mm. However, the predicted subgrade modulus model settlement was low, at approximately 65%. The first stage settlement was fast (7.5mm/1000cycles) which resulted in the difference between the experimental measurement and the predicted measurement, informed by the subgrade modulus model. The track experienced repeated flooding events which changed the soil properties. The flooding reduced the entire subgrade stiffness and strength, particularly at the bottom layer. Furthermore, the new surface layer was not as overconsolidated as the rest of the subgrade.

In comparison with the initial phase, the track settlement in this phase was significantly higher (by a factor of 5) than the initial dry phase; an outcome due to the effect of cyclic wetting and drying, which changes the soil properties. In addition, the bottom section of soil moisture content was approximately 23%, and the degree of saturation approximately 94%. Soil suction also decreased to approximately, 300kPa. The impact of wet and dry cycles on soil behaviour and track performance is discussed in Chapter 6.

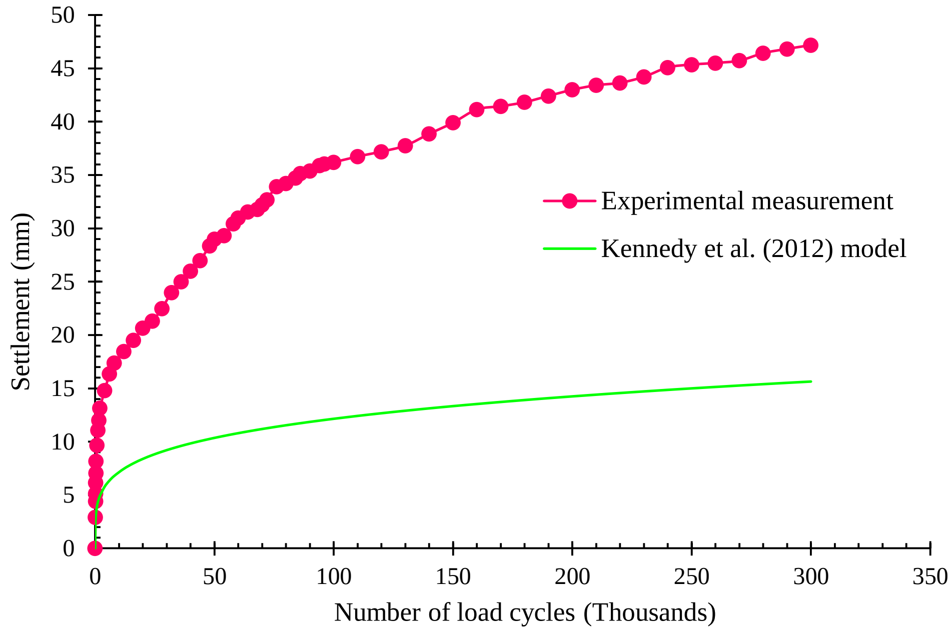


Figure 5.18 Track performances in dry condition after repeated flooding at Phase-III

5.6 Concluding remarks

Sand blanketing is an effective system of curing the subgrade problems including subgrade erosion, slurry formation, mud pumping, attrition and other water related issues. Sand blankets, however, can cause significant settlements problems to the entire railway structure. Ballast is a very permeable layer where clay is a low permeable (depends on soil type and history). In order to drain the water quickly from the structure a granular material layer has traditionally been placed between ballast and clay. Most conventional railway tracks are built by placing ballast on subgrade. However, these types of track face significant problems, especially regarding enhanced maintenance costs (Duong et al., 2014); therefore, use of a sand blanket is a popular method to alleviate subgrade problems. The results of the experiments show that the sand blanket cannot cure all the water related problems; at some point, it can actually cause significant problems, rather than prevent them. It stops clay migration into the ballast but it moves itself into the ballast, thereby causing poor performance. However, it militates against subgrade erosion and ballast penetration into the subgrade.

Efficient track drainage is very important for long-term track performance and maintenance. Water can become entrapped due to drainage blockage or inefficient drainage. In this case, the worst scenario would be if only a sand blanket is placed as a filter layer. The water from a top direction (precipitation), first enters into the ballast and then flows laterally out of the ballast into the drains or enters the sub-ballast. Such water will drain through ditches or enter into the subgrade. Surface ditch drains can collect water from the ballast and sub ballast; and the subsurface water are needed to collect water flowing through the subgrade and may also be needed to help drain sub-ballast (Selig and Cantrell, 2001). Therefore, it is essential to keep drainage paths clear, the edges of the ballast and sub ballast layers must not be blocked. Ballast, fouled due to saturation can entrap water or can block drainage. Selig and Cantrell (2001) stated that the drainage blockage can cause ballast fouling, ballast pocket formation from subgrade settlement, low permeability at the edge of the ballast, can create ponding next to the track and can cause an incompetent lateral slope on the sub ballast surface. In the current study, even after six weeks drying, the track performance had not significantly improved because of the presence of entrapped water between the ballast and the sand blanket.

Soil suction measurement is very important to enable prediction of the track performance. Repeated flooding damages the surface layer despite using a sand blanket; therefore, a new surface layer needs to be installed. The track performance was not the same as in the initial dry condition and the soil properties changed, due to repeated flooding; hence, poor track performance is the result Subgrade properties are very important; particularly soil suction which is discussed in the next chapter.

CHAPTER SIX - CYCLIC WETTING AND DRYING EFFECT OF SUBGRADE AND TRACK PERFORMANCE

6.1 Introduction

As railway track system comprises of a number of components and it is essential to understand the function and behaviour of each component. Historically, rail-track substructure has been neglected and given far less attention than the superstructure, in spite of its importance in track design. Little information is available regarding the evaluation of subgrade performance (Sattler et al., 1989; Selig and Waters, 1994; Bonnet, 2005). Yet, it is necessary to have an understanding of subgrade conditions and behaviour in order to predict and analyse the track behaviour and solve subgrade related problems. This chapter focuses on overall subgrade behaviour when subjected to cyclic wetting and drying.

Subgrade evaluation and maintenace is both difficult and costly as it depends on several factors including soil type, moisture content, shear strength, stiffness and consolidation (McHenry and Rose, 2012). In addition, poor subgrade and inadequate drainage can cause problems including ballast fouling, ballast pockets and pumping of fine particles. Cyclic loading and train speeds, fine grained soil and low bearing capacity of the formation layer all contribute to subgrade problems. However, repeated loading, excess moisture content and poor drainage lead to subgrade failure (Brough et al., 2003b). Water impacts on each component of ballast, sub-ballast and subgrade), but it is the sub-ballast and the subgrade which experience a larger impact, compared to the ballast layer, as it is a single sized rock (Ghataora et al., 2004; Ghataora and Rushton, 2012).

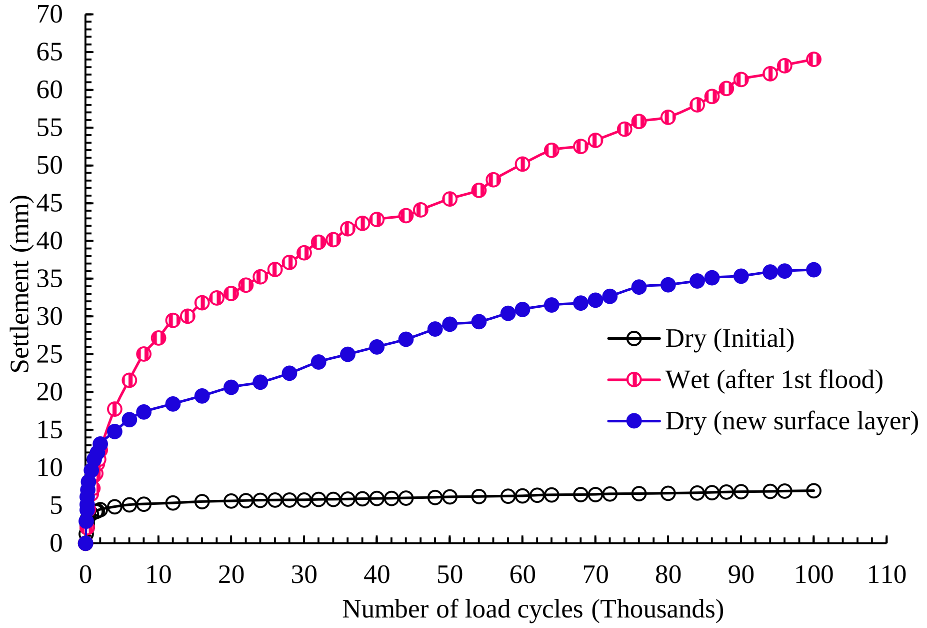
The influence of water on track performance has been discussed in Chapters 4 and 5. It was also noted that traditional sand blanketing neither improved the track performance nor reduced maintenance. However, an important finding was that it is able to mitigate the subgrade erosion problem. Sadeghi and Askarinejad (2009) reported that a reliable evaluation of current track conditions and appropriate predictions of short-term and long-term behaviour are essential for efficient track maintenance. The following sections focus on soil behaviour; in particular, the effect of cyclic wetting and drying on both soil behaviour and track stiffness.

6.2 Track settlement subjected to cyclic wetting and drying

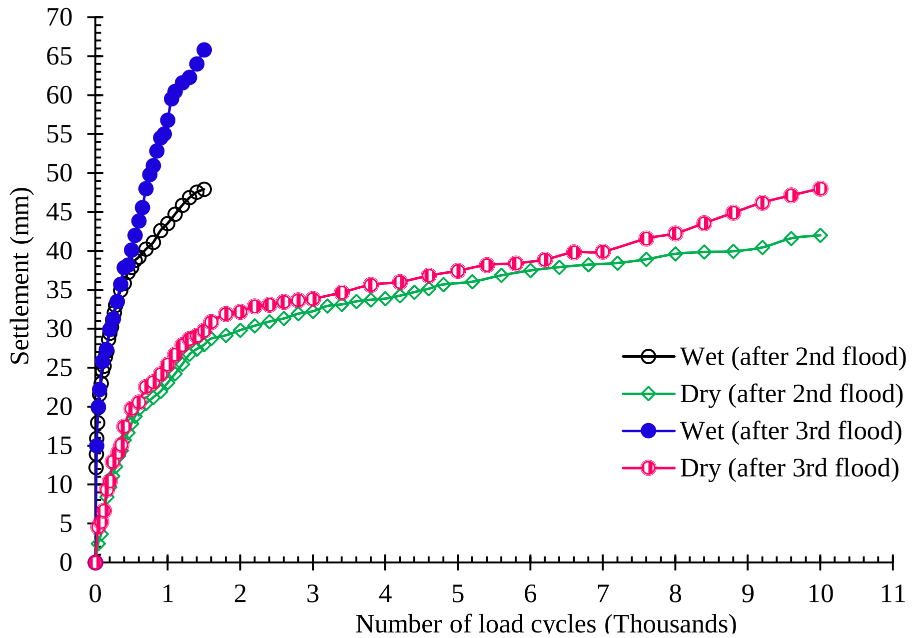
Some portion of ballast can be dry but, in general, sub-ballast and subgrade always contain some amounts of moisture. However, it should be noted that the sub-ballast and subgrade perform best under cyclic load in an unsaturated condition (Selig and Waters, 1994; Duong et al., 2014). Compacted unsaturated subgrade has rarely been investigated on the basis of unsaturated soil mechanics (Gupta et al., 2007; Sawangsuriya et al., 2008; Sawangsuriya et al., 2009).

Figure 6.1a shows the effect of suction on track behaviour in different conditions (unsaturated and saturated). In this case, only three phases were considered to investigate the suction effect and track settlement for 1×10^5 cycles. In other experiments, after 2nd and 3rd flooding, a layer of sand blanket was placed on subgrade; therefore, the track settlement was not considered directly from subgrade. At the beginning of the test, the suction profile was high through the entire depth (see Figure 4.2). The track settlement was low (approximately 7mm) but after the 1st flooding the entire subgrade suction decreased; as a result, track settlement increased significantly, by approximately 65mm after flooding. After a third flooding, the surface layer became significantly soft (subgrade modulus 24.5MPa, see section 6.3.2); on the other hand, the entire subgrade suction profile decreased notably (Figure 5.9); particularly the bottom section of the subgrade. A new surface layer (100mm) was placed, so as to investigate the track performance at dry condition after repeated flooding. Despite the dry surface layer, the track settlement was unexpected (see section 6.3.3 for a predicted track settlement) and high, at approximately 36mm after 10^6 cycles. It was almost 5 times higher than the initial dry phase. This result can be explained as the matric suction of the entire subgrade was not the same as it was in the initial condition. Furthermore, the repeated flooding influenced the entire subgrade soil profile. It is clearly shown that soil suction exerts a significant influence on track settlement. The cyclic wetting and drying behaviour is discussed later in this chapter (see section 6.3.1).

The test run with sand blanketing shows a higher settlement in comparison to a track without sand blanketing; especially after the third flooding (Figure 6.1b). Some water was entrapped between subgrade and the sand layer, which created a water pocket; as a result it severely negatively affected on track performance. The data clearly showed that every occasion of flooding increased track settlement.



(a)



(b)

Figure 6.1 Cyclic wetting and drying effect on track behaviour (a) Experiemnt-1 and (b) Experiment-2

6.3 Hydro-Mechanical behaviour of subgrade

The study of hydro-mechanical behaviour of subgrade soil is important in order to understand the subgrade's behaviour in response to changes in moisture content that are associated with soil suction of subgrade. Drumm et al. (1997) reported that most pavement failures occur as a result of a cyclic wetting and drying effect on subgrade. It is evident that the resilient modulus of the soil decreases with any increase of moisture content, and vice versa (Witczak et al. 2000; Zapata et al. 2007). The water retention characteristics of the soil are intimately related to the matric suction; a relationship which can be explained by the water retention curve (WRC). Matric suction increases and moisture content decreases, resulting in an increase of resilient modulus (Cary and Zapata, 2011). The soil water retention curves were determined by pressure plate and filter paper techniques. The plate load test was conducted to characterize the stiffness of the subgrade.

6.3.1 Water retention curve

The water retention curve is defined by the relationship between volumetric or gravimetric water content, or degree of saturation, or void ratio and soil suction (Fredlund and Rahardjo, 1993; Barbour, 1998; Vanapalli et al., 1999b; Pham et al., 2003; Lu and Likos, 2004; Yang et al., 2004; Péron et al., 2007; Fredlund, 2006; Nuth and L.Laloui, 2011). The WRC measurement is an important tool in unsaturated soil engineering practice (Fredlund, 2006; Péron et al., 2007; Li et al., 2007a; Vanapalli et al., 1999b). The compacted (unsaturated) soil always experiences various wetting and drying cycles and thus suction histories. The behaviour of the unsaturated soil is affected during subsequent wetting and drying cycles, due to the changing of the environmental conditions (Zielinski et al., 2010).

The main purpose of the WRC is to investigate the hydraulic path of subgrade soil. The subgrade experienced wetting and drying due to repeated flooding; it was observed that the track performance also varied at different stages of the cycle. The changes in performance of a track being subjected to wetting and drying were explained by the soil water retention curve. The main concern, in this experiment, was to investigate suction behaviour at a given void ratio and moisture content, which was considered from the test in GRAFT. It was seen in GRAFT that the track behaviour significantly changed after repeated flooding; it was also observed that the track performance was not the same in the dry condition in Experiment-2 at Phase-III.

6.3.1.1 Pressure plate

The drying paths of the WRCs were evaluated from initially saturated samples. After the samples were dried adequately by increasing matric suction, the samples were rewetted by decreasing matric suction to obtain the wetting path. The drying and wetting curves are presented in Figure 6.2. The test was conducted with saturated samples, assuming zero soil suction. The soil sample preparation technique is discussed in section 3.3.1. After increasing the matric suction of the soil sample from zero to the air entry value, the water content remained almost constant. Once the air entry value was passed, further increases of matric suction resulted in significant changes in water content. The water started to be replaced by air after the air-entry value was passed due to the increase in soil suction. The water content steadily decreases to the residual water content level as the matric suction increases beyond the air entry value. (Leverett, 1941; Brooks and Corey, 1964; Fredlund and Rahardjo, 1993; Yang et al., 2004).

After the water drained sufficiently from the sample, any remaining liquid water becomes disconnected and localised at the inter-particle contacts (Fredlund and Rahardjo, 1993; Wheeler and Karube, 1995). At this point (residual matric suction ψ_r and residual water content w_r), the matric suction is only due to the water menisci; therefore, a higher energy is necessary to remove any further liquid from the voids. The changes in water content after the matric suction increased beyond the residual value, was followed by very small changes in water content.

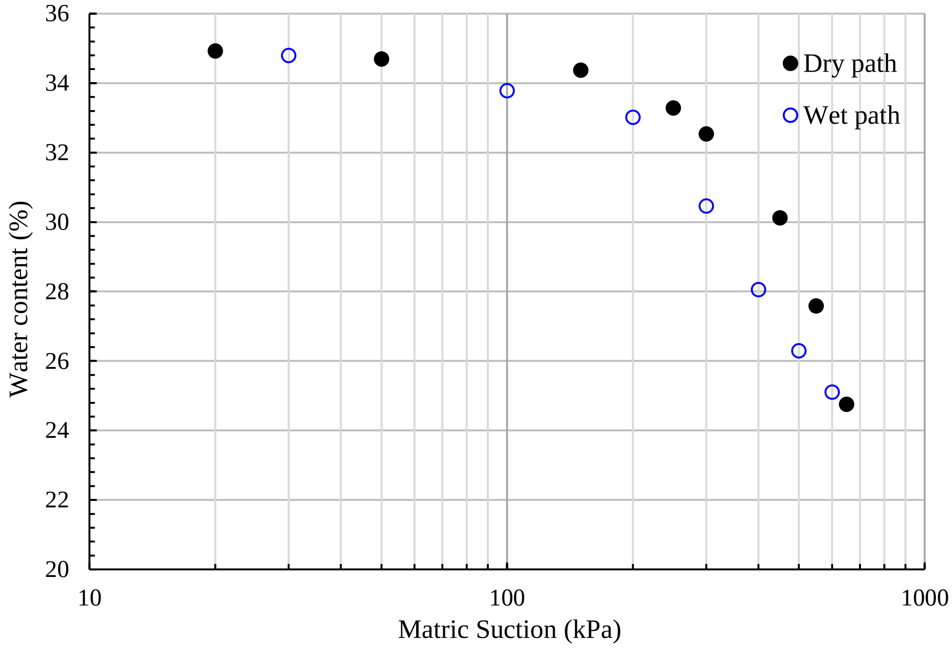


Figure 6.2 Soil water retention curve by the pressure plate technique

6.3.1.2 Filter paper

Supplementary data points were obtained by the filter paper technique. The main reason for this test is to determine the soil behaviour due to the effect of a cycle of drying and wetting. The pressure plate test (using a sample with a hydraulic history) is limited to 650kPa. The compressor in the geotechnical laboratory is only able to give air pressure up to 700kPa; therefore, the filter paper method (without hydraulic history) was used. The results were checked with two different techniques.

The soil samples were prepared in the same way as for the pressure plate test. The sample preparation and experimental techniques are discussed in section 3.3.3. A complete WRC was not evaluated from a single sample but from a number of identical soil samples. Figure 6.3 presents the reproducibility of identical soil samples. Therefore, the notionally identical soil samples did not influence the accuracy of the results.

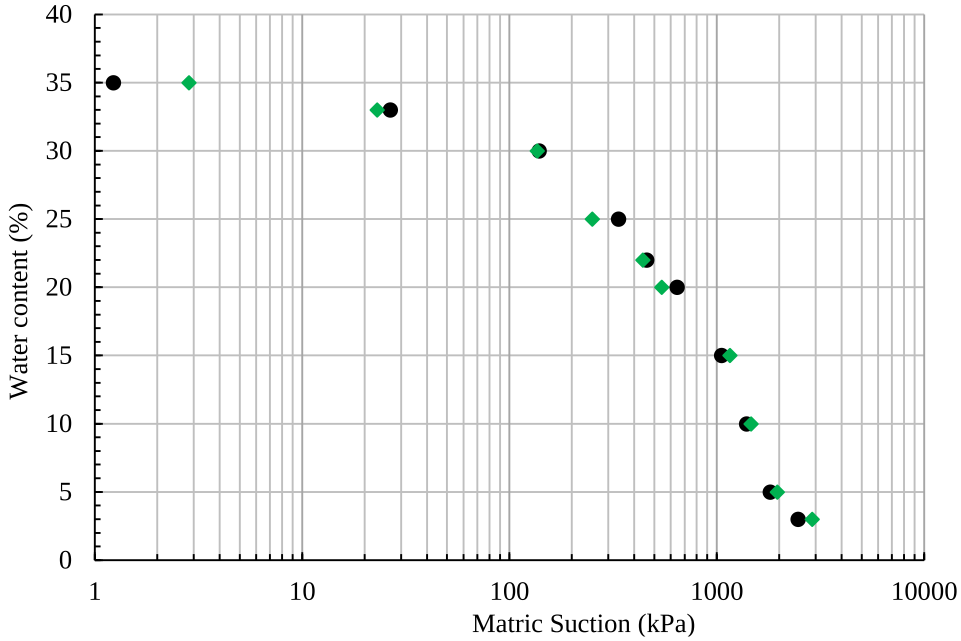


Figure 6.3 Demonstration of the identical soil samples

The saturated samples were allowed to dry in the open air until the desired water content was achieved. The water contents air dried samples were 33%, 30%, 25%, 22%, 20%, 15%, 10%, 5% and 3%. The samples were rewetted to obtain the wetting curve. The laboratory humidity chamber was not available at that time; therefore, the samples were placed on a 3/4 submerged brick in an aluminium container, covered by another aluminium container, to create an artificial environment with which to rewet the sample. Silva et al., (2008) also used a similar technique to rewet their sample whereby they placed their sample on top of porous stone in a vase with water. Figure 6.4 shows the hydraulic path obtained by use of the filter paper method. The drying path and the wetting path were also different from what was seen in the pressure plate test. Figure 6.5 presents combined data obtained from the filter paper and pressure plate tests for the drying and wetting paths. The suction values were different between pressure plate and filter paper methods, particularly at low suction values. Chandler et al. (1992) used the pressure plate to calibrate low suction values; there were significant variations between 5kPa up to 40kPa. In addition, smearing of clay on the filter paper at low suctions can lead to an overestimation of the filter paper's weight. Furthermore, differences between the calibration curves may have shown different values, due to varying equilibrium periods. Nam et al. (2010) reported from their six different techniques that the results do not show perfect agreement for different suctions and degree of saturation. However, the results appear to be comparable and the scatter in the results appears to be within the range expected from the sample variability. Noguchi et al. (2011) also found the differences between the filter paper method and the pressure plate method. The SWCC obtained by the pressure plate method seemed to overestimate the water content. Two possible reasons were considered to explain such differences between the pressure plate and the filter paper method: a) the way water is expelled from the soil sample and b) the contact between the soil sample and the ceramic plate. Within a soil sample which is drying in the atmosphere, the water in the pores cavitate due to soil suction and move towards the surface of the sample to evaporate. On the other hand, when the soil sample is in the pressure chamber, the water pressure is maintained above atmospheric pressure. The calibrated van Genuchten's models for the SWCC are shown, and appear to provide a good fit with the experimental results (as shown in Figure 6.8).

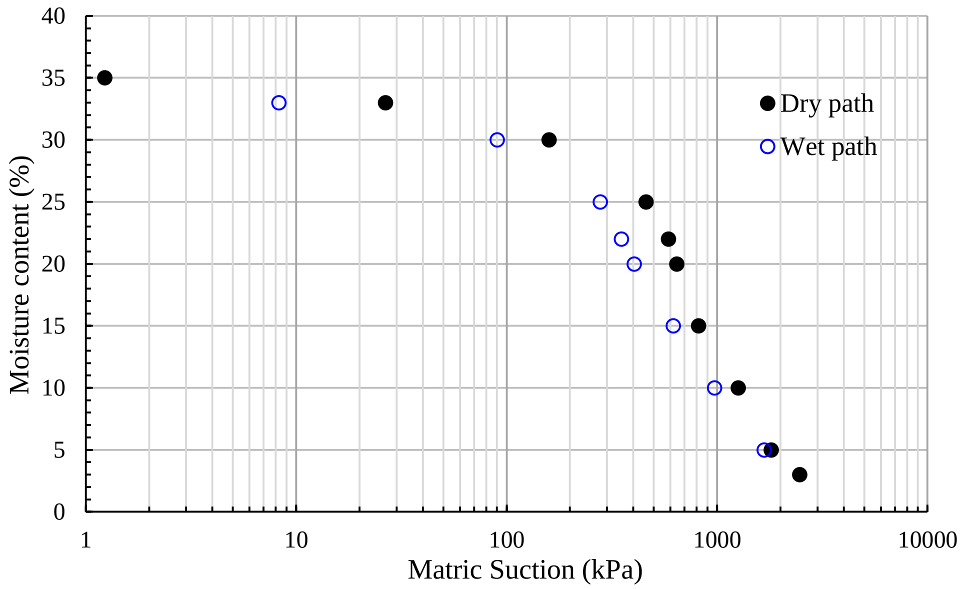


Figure 6.4 Drying and wetting path filter paper technique

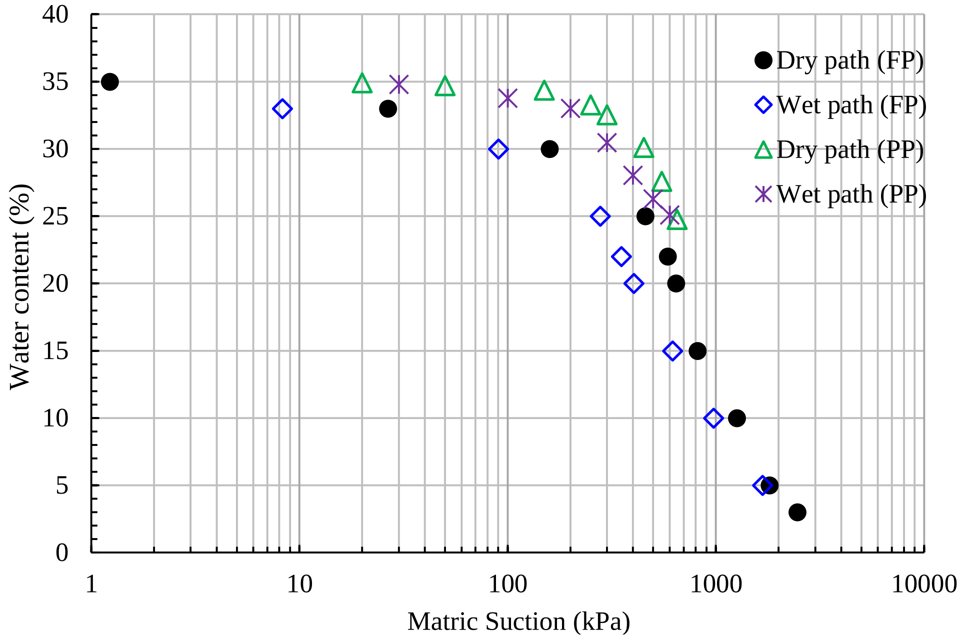


Figure 6.5 Summary of all data obtained by pressure plate and filter paper

6.3.1.3 Mathematical description of the WRC

The experiments of WRC have limitations regarding the amount and range of suction measurement; hence, to predict the soil behaviour, (i.e. flow behaviour) a mathematical equation is necessary (Fredlund and Xing, 1994; Lu and Likos, 2006). For the analysis and demonstration of hysteresis of the WRC, the fitted equation of van Genuchten (1980) was used, as shown in Figure 6.6 for the pressure plate test and 6.7 for the filter paper technique. Figure 6.8 shows the fitted curve for the combined data obtained from the pressure plate and filter paper tests. There are a number of equations available, but in this research there is no intention to evaluate all the various WRC equations. In semi-logarithmic scale, hysteresis generally decreases as the water content is decreased, following an increase in matric suction. Table 6.1 shows the parameters for the curve fitting.

Table 6.1 van Genuchten (1980) curve fitting parameters

Model	Formula	Technique	Drying		Wetting	
			α	n	α	n
van Genuchten	$\theta = \theta_r + \frac{\theta_s - \theta_r}{[1 + (\alpha\psi)^n]^m}$	PP	0.0016	3.40	0.0021	2.67
		FP	0.0015	3.70	0.0028	2.20
		PP & FP	0.0015	4.30	0.0032	2.90

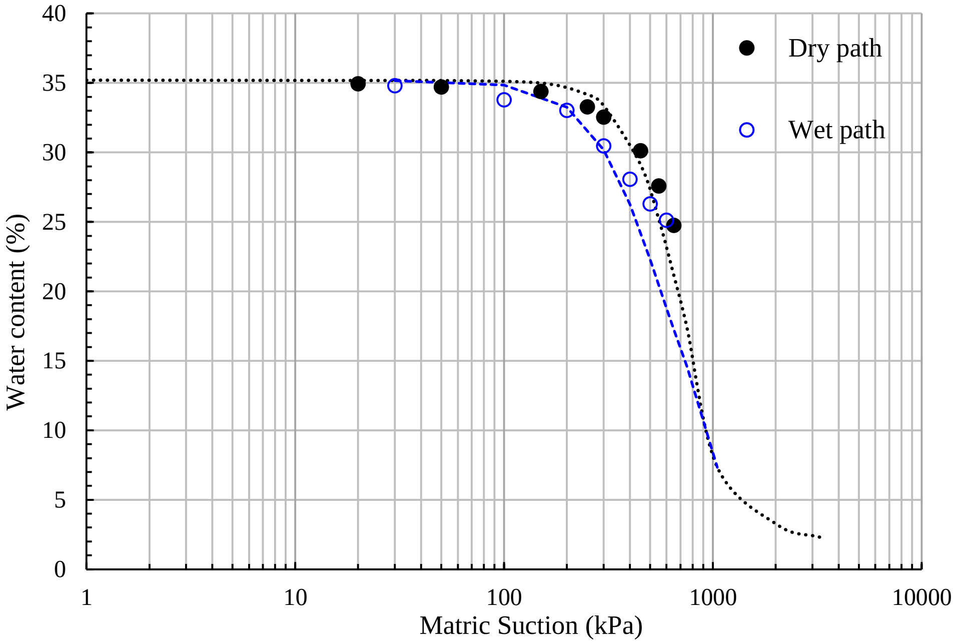


Figure 6.6 Experimental WRC by pressure plate along with van Genuchten fitted curve

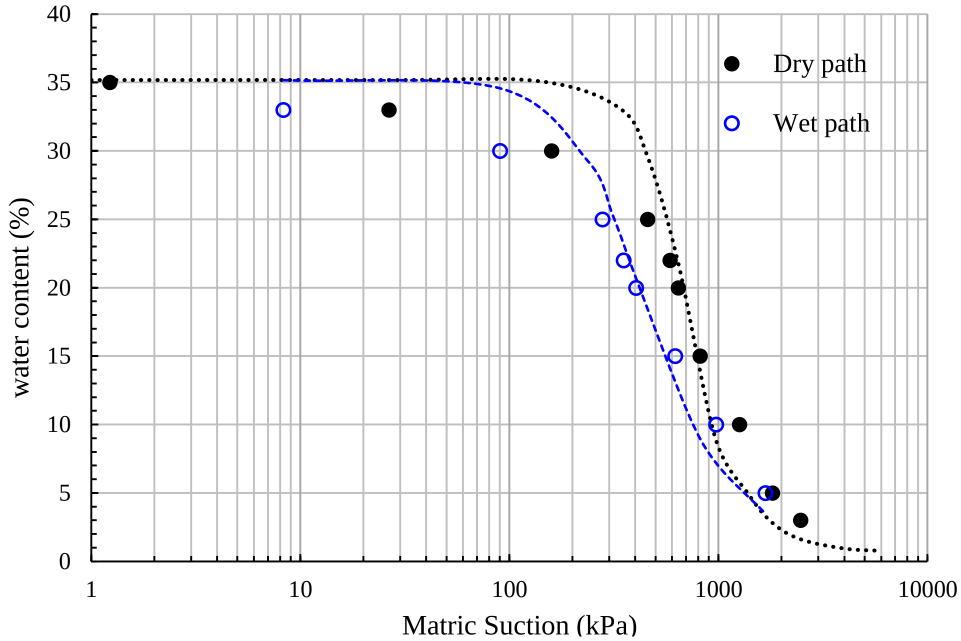


Figure 6.7 Experimental WRC by filter paper with van Genuchten fitted curve

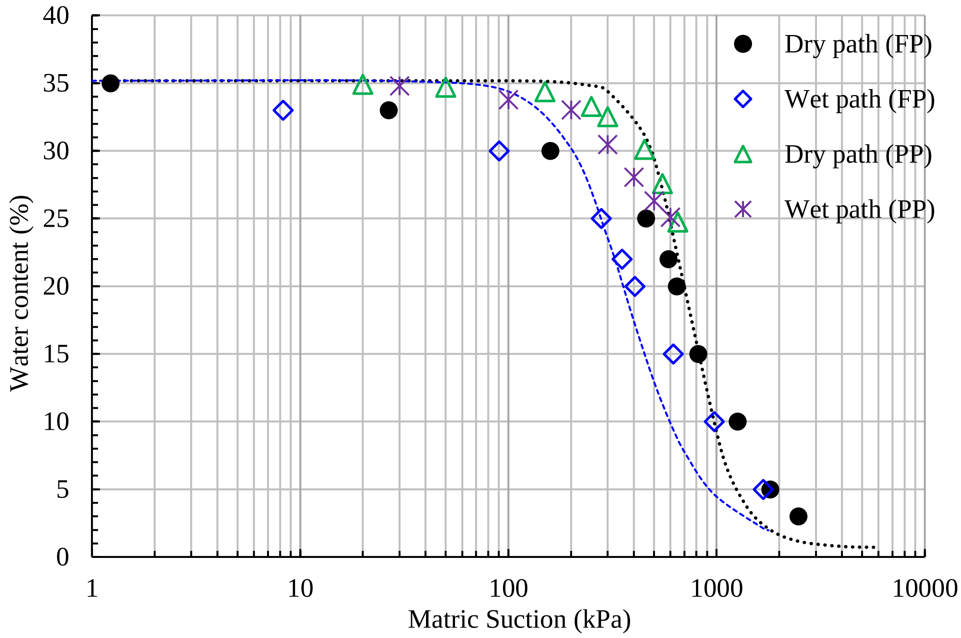


Figure 6.8 Experimental WRCs with van Genuchten fitted curve

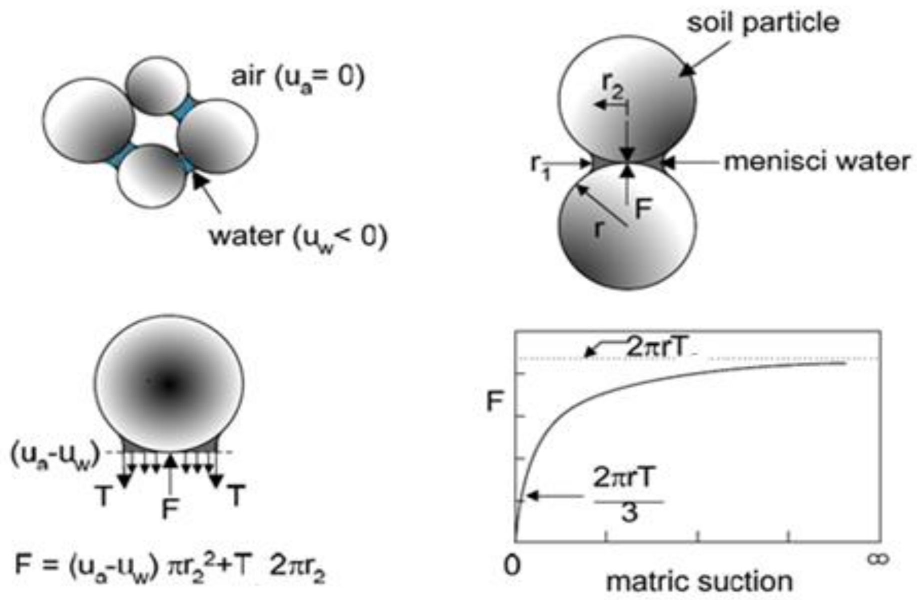
6.3.2 Influence of matric suction on stiffness

Stiffness is an important subgrade parameter as it measures the ability of soil to resist deformation. It characterises the structural condition of the track support (Ebersohn and Selig, 1994; Priest and Powrie, 2009). The railway subgrade stiffness and soil performance in general, is governed by two characteristics: strength and deformation (Selig and Waters, 1994; Brough et al., 2006; McHenry and Rose, 2012). Hunt (2000) reported that the track quality, track performance and subsequent maintenance are highly dependent upon the magnitude and variation of subgrade stiffness.

Recent research substantiated the importance of track bed stiffness and its relationship with geometrical deterioration, track performance and maintenance (Brough et al., 2006). Hunt (2005) stated that the subgrade properties are the primary determinants of overall track stiffness. Ebersohn and Selig (1994) documented that the measurements of substructure stiffness magnitude and variation signify the expected amount of track roughness development and maintenance requirement. Vertical track stiffness is a combination of all the substructure and superstructure stiffness; the local stiffness of each layer determines the displacements of each layer, but the substructure gives support to all the components, including substructure and superstructure (Berggren, 2009). If the track stiffness changes along the track it can induce vehicle track interaction dynamic forces; as a result, such changes can cause differential settlement and potentially other problems (such as reduced life of components and increased maintenance cost). Lower stiffness can cause high ballast strains, therefore ballast settlement (Hunt, 2005; Banimahd, 2008). Berggren (2009) reported that the present understanding of the issue of track stiffness, and its effect on track performance, is inadequate; in fact, currently there is no European standard for vertical track stiffness. It has been mentioned earlier that the lack of correlation between maintenance, track quality deterioration and subgrade condition is rarely considered in the design and implementation of railway tracks. Track maintenance work is generally undertaken without giving priority to the subgrade, although subgrade stiffness is an important factor to take into account when trying to evaluate the track performance and maintenance work. Subgrade modulus or resilient modulus, is the behaviour of the stress-strain of subgrade soil under traffic loading. It has been widely used to design and analyse substructure.

The mechanical behaviour of unsaturated soils is heavily influenced by moisture and suction change has been recognized in the area of geotechnical design, particularly the behaviour of compacted subgrade soil (Gupta et al., 2007). Therefore, the stiffness of compacted soils can be expected to change in response to changing moisture conditions and suction. Thus, knowledge of the stiffness-suction-moisture content relationship of compacted soils is essential in order to understand the subgrade behaviour during construction and subsequently when the track is in service. Khoury et al. (2003) observed that higher soil suction generates higher resilient modulus. Dawson and Correia (1996) also noted stiffness increased when matric suction increased. In addition, subgrade soil undergoes cycles of drying and wetting which influence mechanical behaviour. In previous sections of this thesis, the behaviour of soil that was subjected to dry and wet condition cycles was discussed. Wheeler and Karube (1995) reported that the soil experiences both drying and wetting but generally shows different mechanical behaviour during drying in comparison with the same value of suction during wetting. Sharma (1998) suggested that compacted unsaturated soil shows a different stiffness during initial loading at the same constant suction, depending on whether the sample undergoes a wetting–drying cycle.

Fisher (1926) proposed a simple model to understand the behaviour of stiffness increase due to matric suction and how a water air meniscus affects the stress state of the unsaturated soil; as presented in Figure 6.9a. The meniscus water at the point of spherical particle contact generates a force (F) normal to the plane passing through the contact point, and orthogonal to the line connecting the particles' centres. This force only arises from menisci water and increases as suction increases. Therefore, the effects of matric suction result in a greater than normal force holding the particles together and greater slippage-limiting strength. Consequently, the unsaturated particulate media show a stiffer and more resistant to load response, as an example of higher shear strength, with respect to that of dry contacts or fully saturated particles. Depending on the size of the particulate media, and thus their corresponding pore sizes, the stiffness and strength of unsaturated particulate media increase with increasing matric suction. However, this effect does not increase infinitely as the contact force (F) tends towards a limiting value, due to the progressive reduction in the meniscus radius as suction increases (as shown in Figure 6.9b).



(a) (b)

Figure 6.9 Water-air menisci between two solid spheres: (a) impact of suction on the normal force between the spheres and (b) induced normal force versus suction (after Mancuso et al., 2002)

Stiffness of soil is an important engineering property, particularly for subgrade design and analysis. Past studies have focused primarily on the stiffness and modulus-moisture relationship of compacted soils in the as-compacted state (Li and Selig, 1994; Muhanna et al., 1999; Ooi and Pu, 2003; Yuan and Nazarian, 2003). Comparatively, few studies have been reported regarding relationships between modulus and suction, corresponding to in-service moisture changes (Fredlund and Morgenstern, 1977; Gupta et al., 2007; Khoury and Zaman, 2004). Furthermore, it is difficult to quantify the relationship between moisture content and resilient modulus, because the relationship is highly dependent on soil type. Therefore, some researchers have interpreted measured data in terms of matric suction rather than moisture content as the resilient modulus increases with increasing matric suction (Fredlund and Morgenstern, 1977; Khoury and Zaman, 2004; Yang et al., 2005; Yang et al., 2008). Generally, the subgrade modulus test is conducted in the laboratory, based on different moisture content, where matric suction is unknown.

Figure 6.10 presents the obtained data from the test at different phases along with matric suction. Only 5 plate load tests were performed (plate load test techniques are discussed in section 3.2.3). Initially, the results of the plate load test showed the track bed to be very stiff as the matric suction was high (1300kPa). The subgrade tangent modulus was approximately 109MPa and the subgrade reloading modulus was approximately 122MPa. At the end of Experiment-1 (after 1st flooding), the subgrade tangent modulus was 30MPa and subgrade reloading modulus was 35MPa. The soil matric suction in the surface layer was 150kPa. The subgrade surface suction reduced after the 2nd and 3rd floodings; consequently, the stiffness of the subgrade also decreased. The subgrade moduli were around 25MPa and 24MPa and subgrade reloading moduli were approximately 32MPa and 29Mpa respectively. After a 3rd flooding, it was decided that the surface layer needed to be replaced as it had become significantly soft, also the drying phase behaviour after frequent flooding needed to be investigated. This time the matric suction was approximately 700kPa. The subgrade modulus was approximately 78MPa and the subgrade reloading modulus was approximately 94MPa. The entire subgrade moisture content increased and suction decreased, due to the cycle of drying and wetting. These findings clearly indicate that the suction has a great influence on subgrade stiffness. The research suggests that the subgrade suction measurement is an important parameter for track subgrade design and assesment of maintenace work. The summary of the results are presented in Table 6.2.

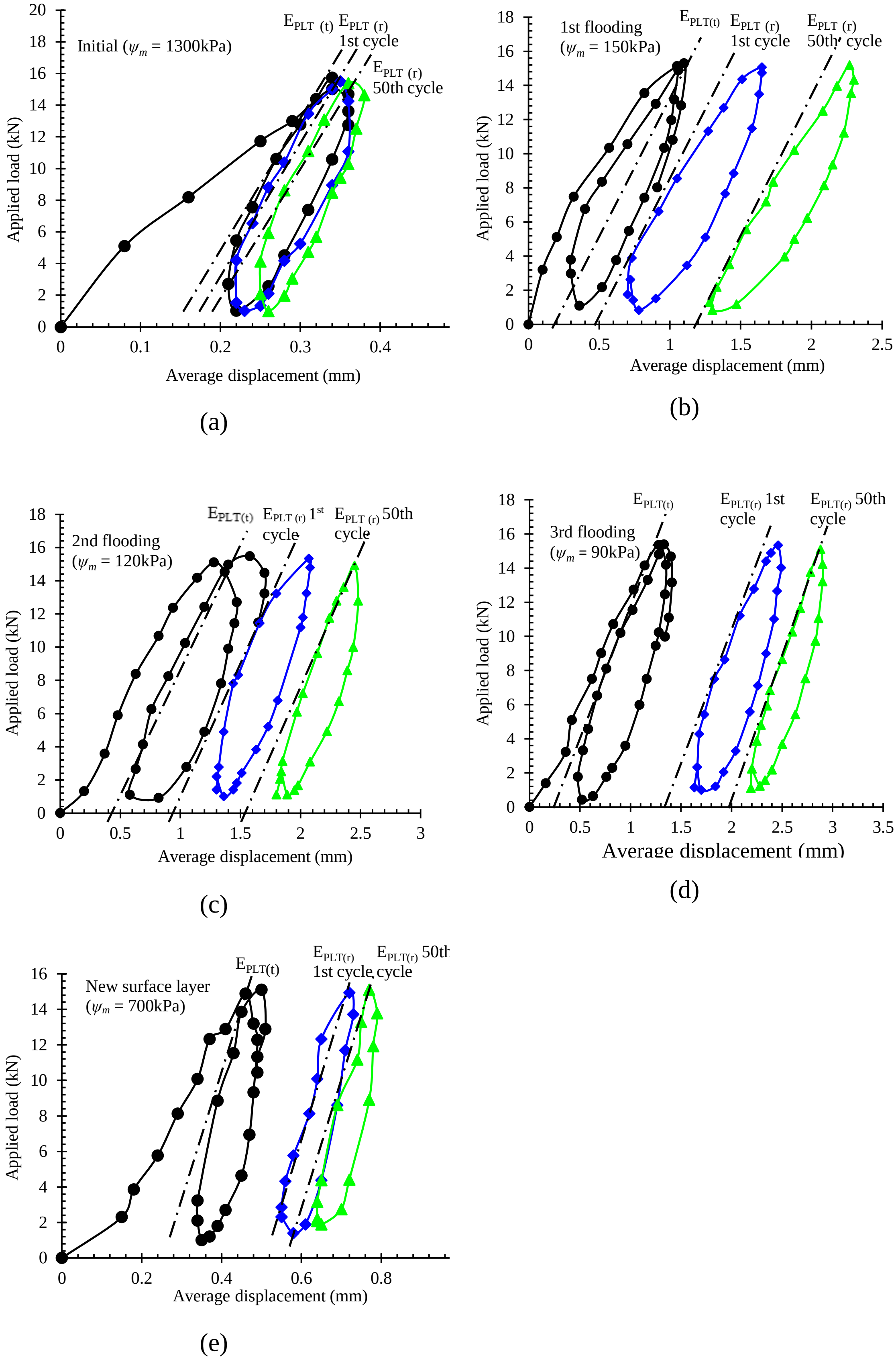


Figure 6.10 Typical plate load test results at different phases

Table 6.2 Summary of subgrade modulus at various phases

Test	Formation Moisture content, w (%)	Formation Matric suction (kPa)	Subgrade tangent modulus (MPa)	Subgrade reloading modulus (MPa)
Initial	10.05	1300	109	122
1st Flooding	27.48	150	30	35
2nd Flooding	31.01	120	25	32
3rd Flooding	31.34	90	24	29
New surface layer	14.00	700	78	94

Figure 6.11 shows the relationship between degree of saturation and matric suction in the subgrade's surface layer (100mm) before the cyclic loading test. Figures 6.12 and 13 present the variation of subgrade modulus with change of moisture content and matric suction, irrespective of whether it is following a wetting or drying path. The experiment's results showed that the subgrade modulus (both tangent and reloading) increased with increases in matric suction and decreases in moisture content. It can be seen from both Figures 6.12 and 6.13 that the subgrade tangent and reloading modulus decreased by 78% and 74% when the moisture content increased from 10% to 27%. The matric suction decreased by approximately 88%. In Experiment-2, Phase-III, the subgrade tangent and reloading modulus increased by almost 238% and 293% when the moisture content decreased from 31% to 12%. The matric suction increased tenfold (70kPa to 700kPa). When a soil specimen becomes unsaturated, voids are partially filled with water and partially occupied by air, resulting in an air-water interface in each void. When the matric suction increases, the radius of an air-water interface decreases and therefore induces a larger normal interparticle contact force (Fisher, 1926; Mancuso et al., 2002; Wheeler et al., 2003; Ng et al., 2013). The experimental data clearly indicate that with the moisture content of subgrade soil approaching saturation, a sharp decline in subgrade modulus can occur due to the low matric suction in the subgrade.

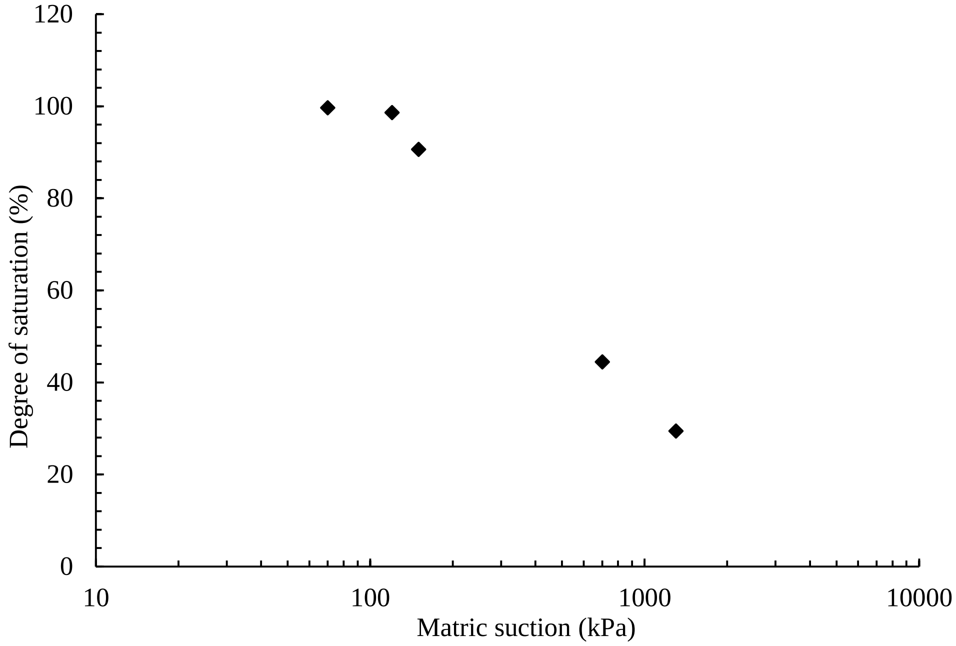
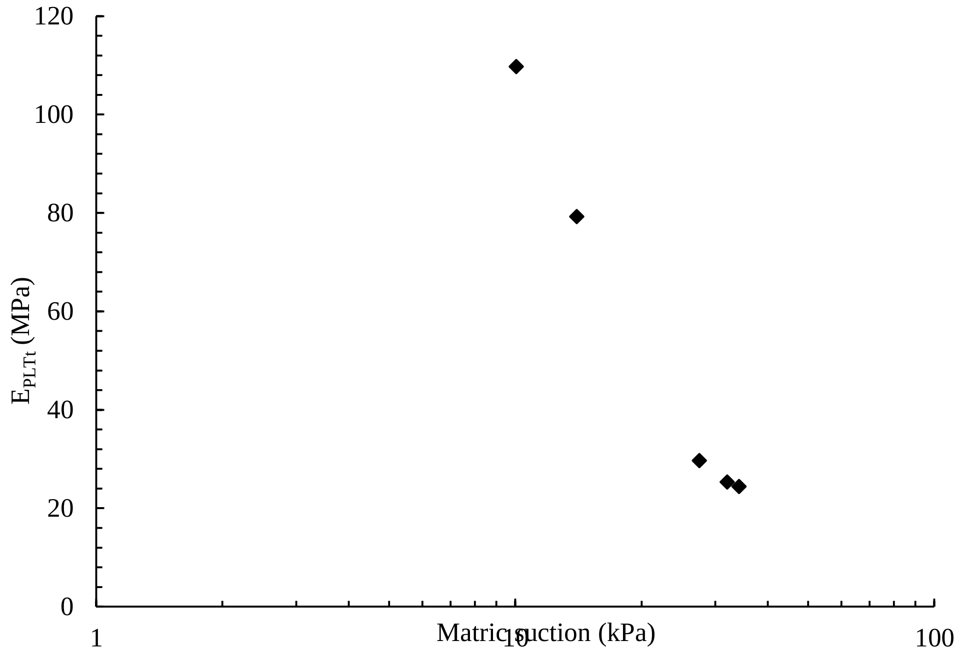
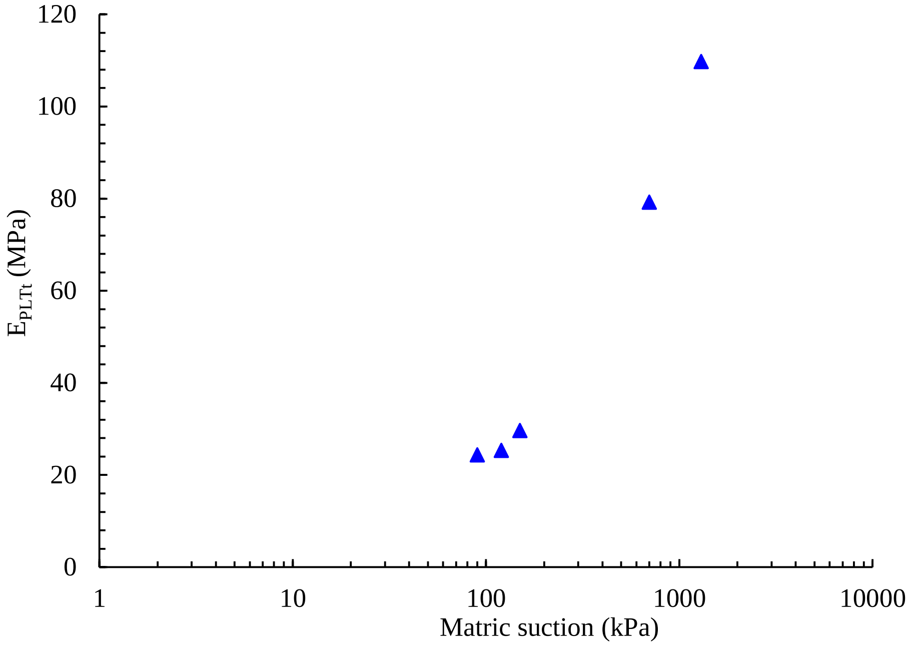


Figure 6.11 Relation between matric suction and degree of saturation

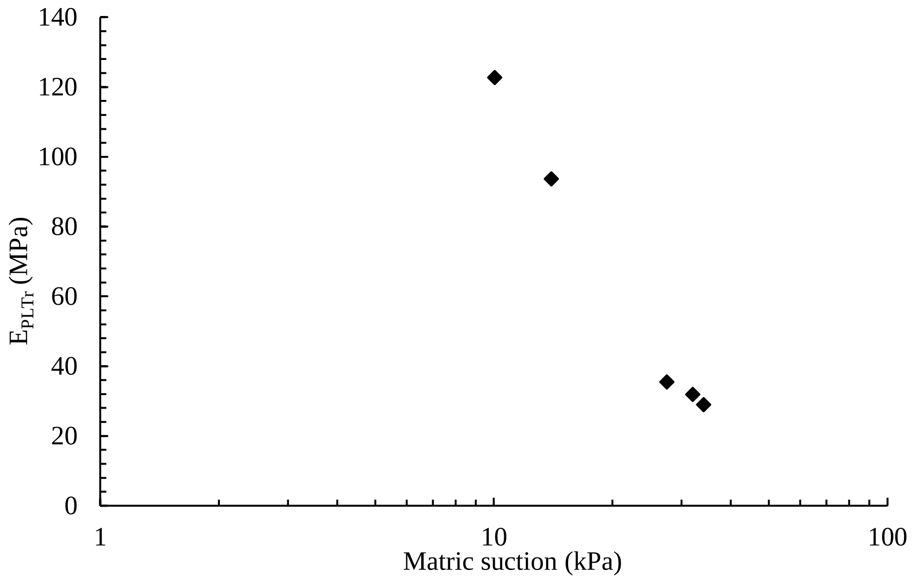


(a)

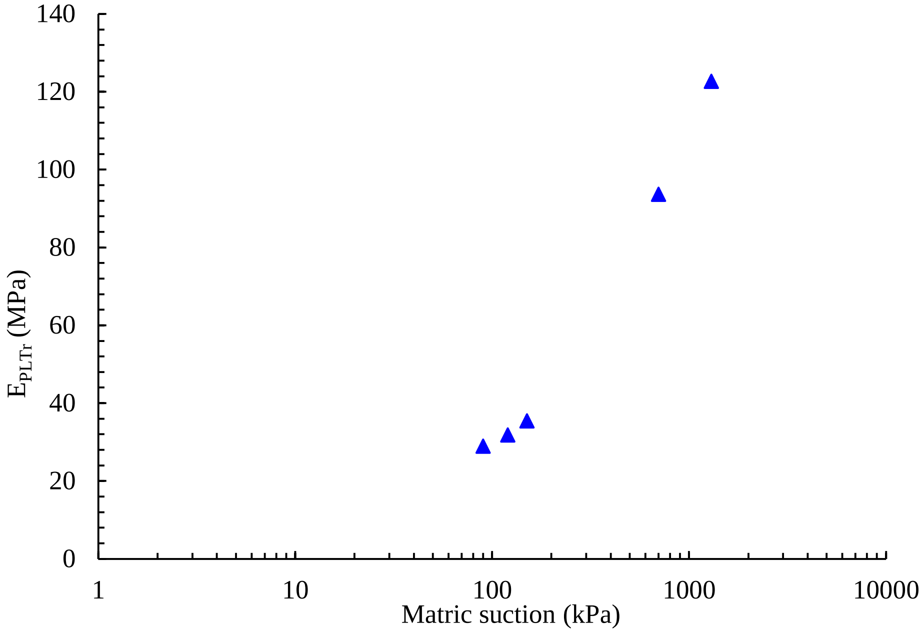


(b)

Figure 6.12 Variation of subgrade tangent modulus with changes of (a) moisture content and (b) matric suction



(a)



(b)

Figure 6.13 Variation of subgrade reloading modulus with changes of (a) moisture content and (b) matric suction

A semi-logarithmic relation between subgrade modulus and matric suction is developed based on the test data present in Figures 6.14 and 15. The equations are given below:

$$E_{PLTt} = 32.52\log(\psi) - 128.5 \quad (6.1)$$

$$E_{PLTr} = 36.12\log(\psi) - 139.8 \quad (6.2)$$

where E_{PLTt} = subgrade tangent modulus, E_{PLTr} = subgrade reloading modulus and ψ = matric suction. These two proposed equations have been developed based on GRAFT subgrade soil (kaolin clay). It is only able to give subgrade modulus based on matric suction.

The Equation 6.2 can be converted to resilient modulus according to (AASTHO, 1993) as follows:

$$M_r = 700 \log (\psi) - 2712 \quad (6.3)$$

where M_r = resilient modulus and ψ = matric suction

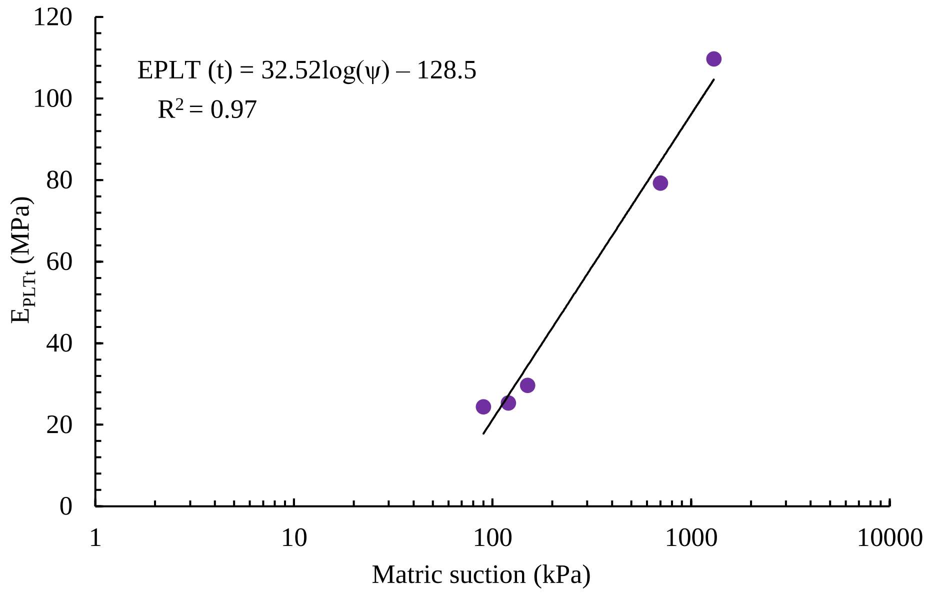


Figure 6.14 A linear semi logarithmic relationship between matric suction and subgrade tangent modulus

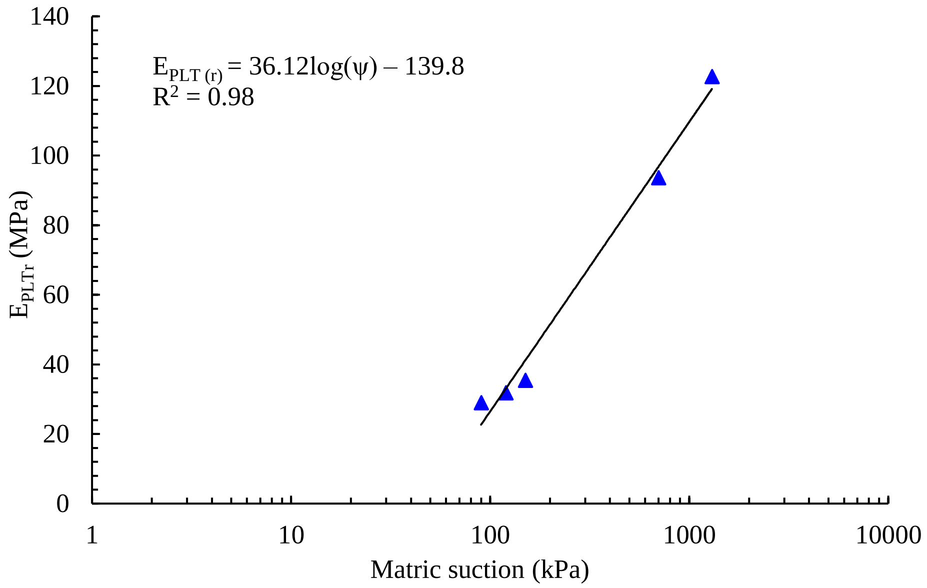


Figure 6.15 A linear semi logarithmic relationship between matric suction and subgrade reloading modulus

Figure 6.16 presents experimental and predicted data (Equation 6.1 and 6.2) for both subgrade tangent and reloading modulus in different subgrade conditions. The subgrade modulus decreased significantly after 1st flooding; however, after 2nd and 3rd flooding, the subgrade modulus did not show any significant difference. Because, the inter-particle voids, which were previously occupied by air during wetting, may have remained water-filled. After replacing the surface layer, the subgrade modulus increased. The track was allowed to dry for 8 weeks, during which time the track was reaching in equilibrium state.

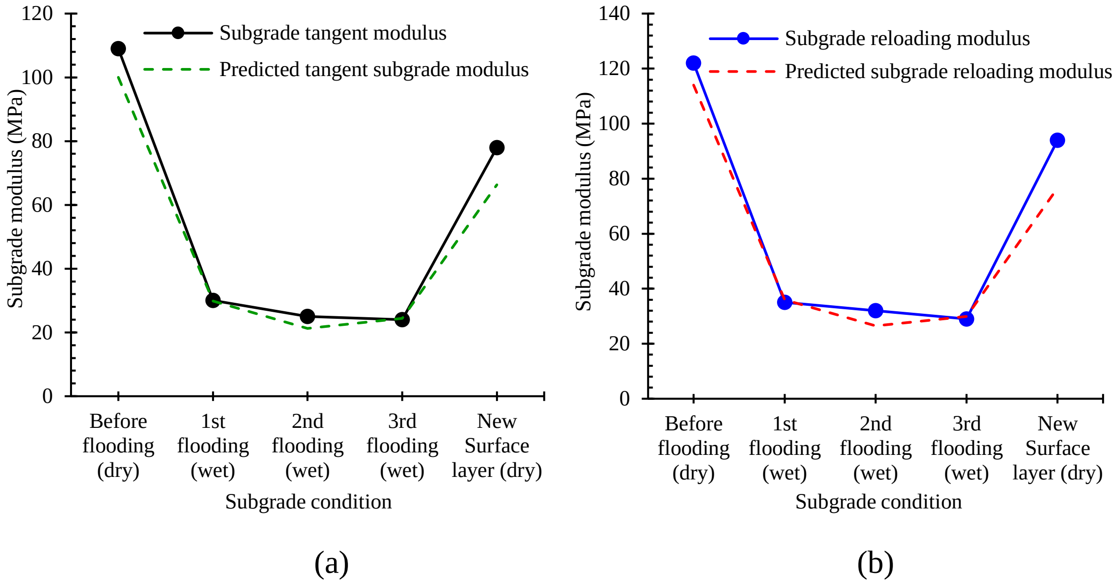


Figure 6.16 Experimental and predicted measurement of subgrade modulus in different subgrade condition (a) Tangent modulus and (b) reloading modulus

Gupta et al. (2007) performed a series of bender element tests on subgrade soil to measure the resilient modulus (M_r). The samples were collected from four different regions of Minnesota. The soils were: i) a silty soil from Red Wing, ii) a silty clay loam soil from Red Lake Falls, iiI) a loam soil from Mn/ROAD facilities near Monticello, and iv) a clay soil from TH 23 near Duluth. Clay content of these soils varied from 4.8-75.2%. Resilient modulus was calculated based on both external and internal displacement measurement and at a bulk stress of 83 kPa and octahedral shear stress of 19.3 kPa. Gupta et al. (2007) proposed a similar type mathematical model to predict the resilient modulus (Equation 6.4). Figure 6.17 shows the relationship between the resilient modulus (M_r) and the matric suction. However, Gupta's mathematical model overestimates subgrade modulus; particularly, in the case of lower matric suction.

$$M_r = 57898\log(\psi) - 54105 \tag{6.4}$$

where M_r = resilient modulus and ψ = matric suction

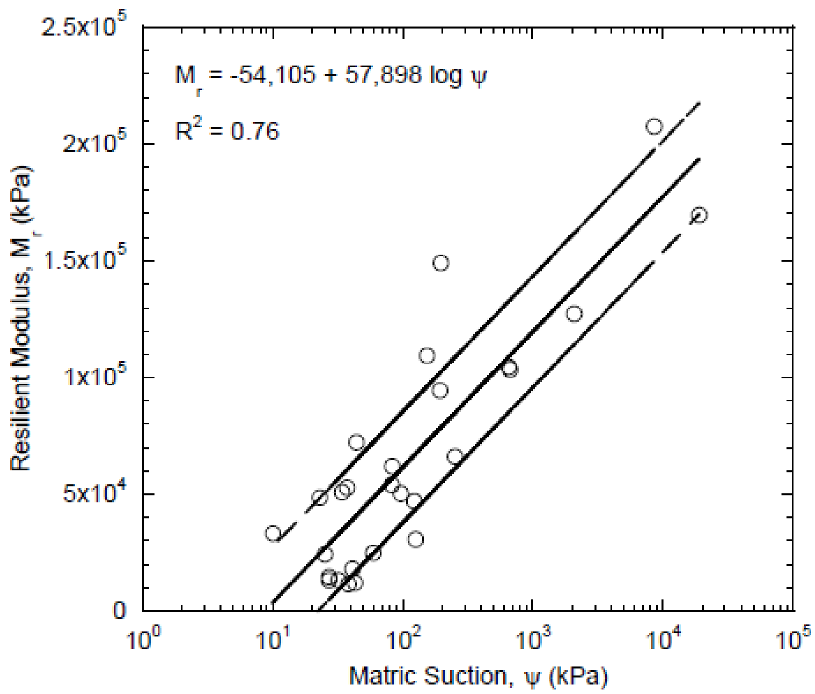


Figure 6.17 A linear semi-logarithmic relationship between Mr and matric suction (Gupta et al., 2007)

6.3.3 Track settlement modelling based on soil suction

Kennedy et al. (2012) proposed a model (Equation 2.7) for predicting settlement in the GRAFT. The effect of subgrade soil structure factors such as fabric anisotropy, inter-particle bonding and degradation of bonds on the behaviour of the subgrade was not considered in the model. The model can predict the track settlement appropriately in dry condition (when track subgrade is following a drying path); however, in the case of wet condition (when track subgrade is following a wetting path), the model under-predicts the settlement. A new equation is proposed that is based on a subgrade wetting and drying path. The subgrade wetting and drying path behaviour is discussed in section **6.3.1.** The proposed new equation is expressed as follows:

$$\text{Track settlement } y = aN^b \tag{6.5}$$

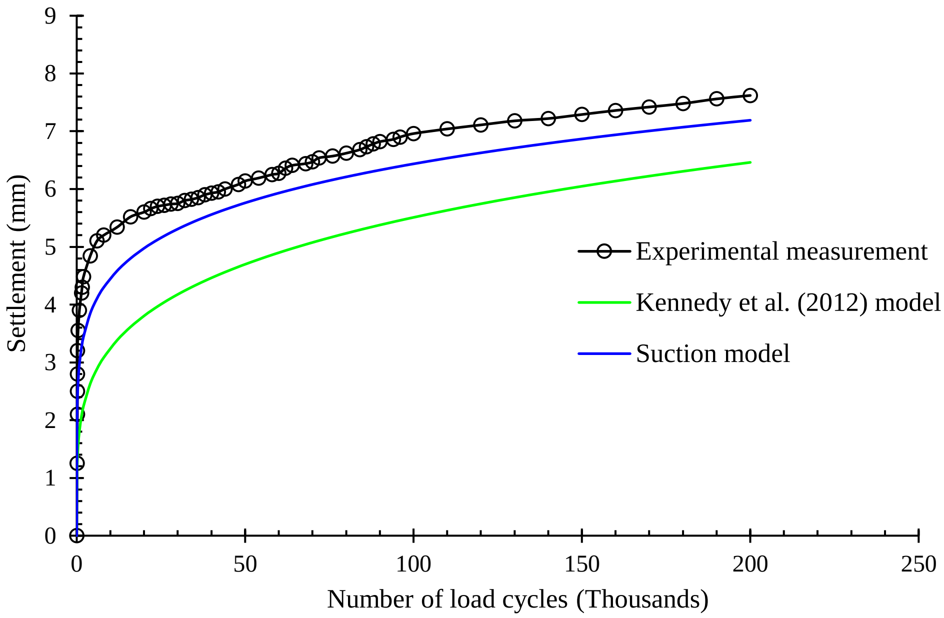
where

For the drying path, $a = 1.024\, e^{0.094S_1}$ and $b = -0.093\log(\psi) + 0.79$ (6.6)

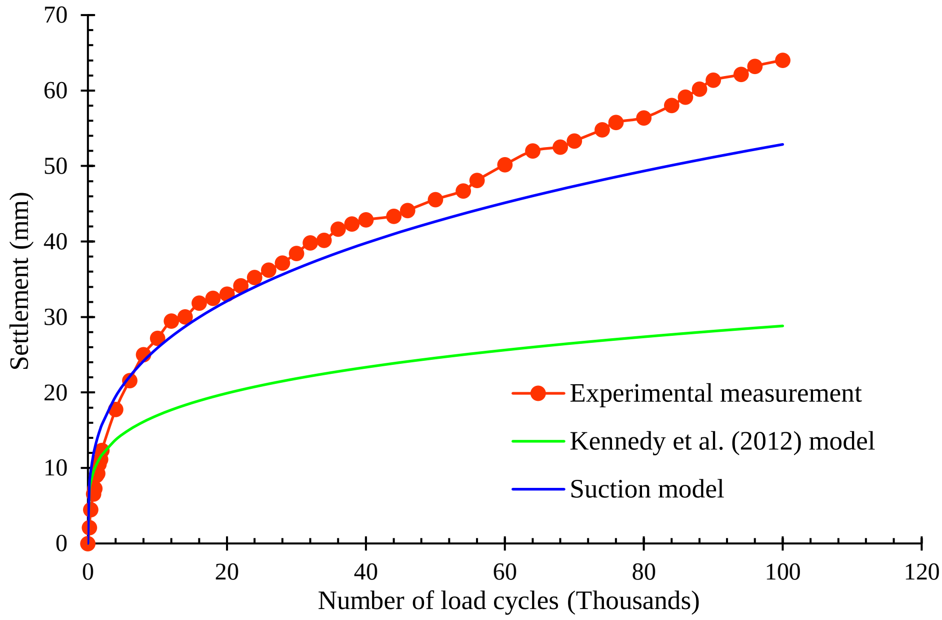
For the wetting path, $a = 1.032\, e^{0.095S_1}$ and $b = -0.056\log(\psi) + 0.56$ (6.7)

Here, S_1= settlement at first cycle and ψ = matric suction.

Figure 6.18 presents a comparison between the proposed model and the model from Kennedy et al. (2012), for the wetting and drying conditions. The Kennedy et al. (2012) model shows approximately similar predictions in drying condition (Figure 6.17a). However, during the wetting path (after one week flooding), the Kennedy et al. (2012) model under-predicts the track settlement (Figure 6.17b). After, repeated wetting and drying, the surface layer was replaced and allowed to air-dry the subgrade, in order to investigate the cyclic wetting drying effect on track performance. In this case, the Kennedy et al. (2012) model under predicts the track settlement (Figure 6.17c). However, the proposed model also under predicts the track settlement. As mentioned earlier (Section 5.5) the surface layer was not overconsolidated as this layer was comparatively young than the rest of the subgrade. Therefore, the experimental measurement is higher than the predicted settlement.



(a)



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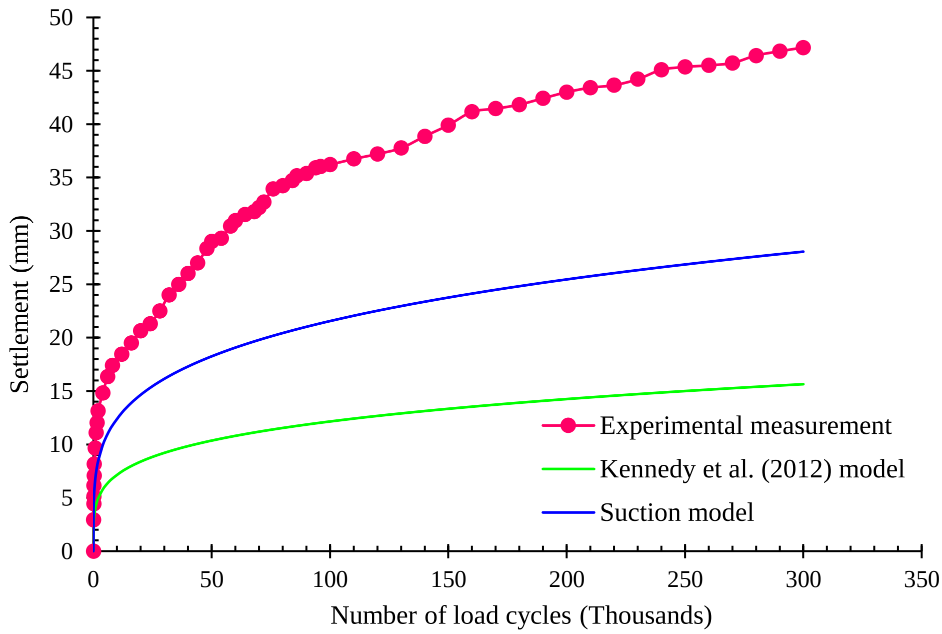
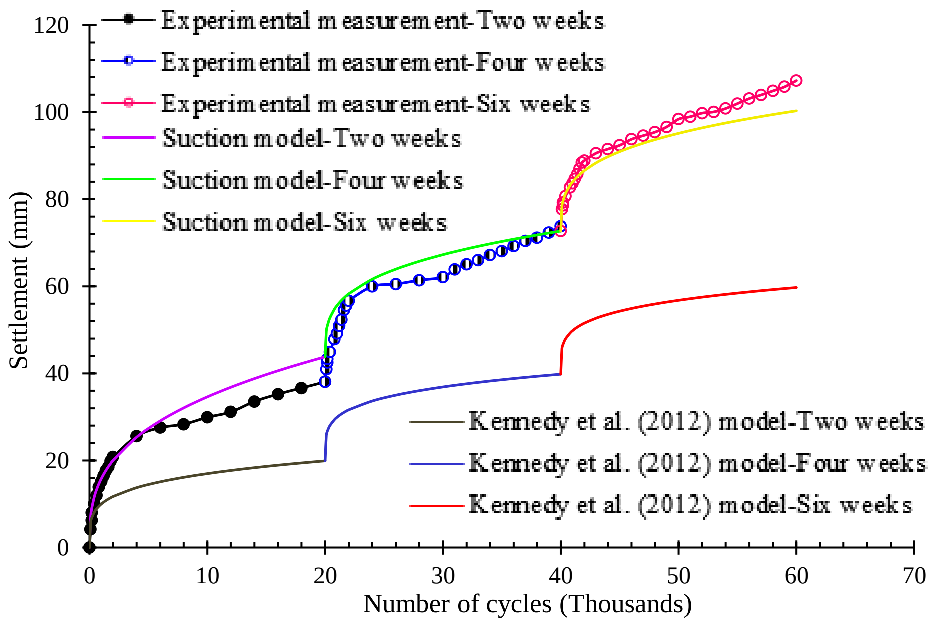


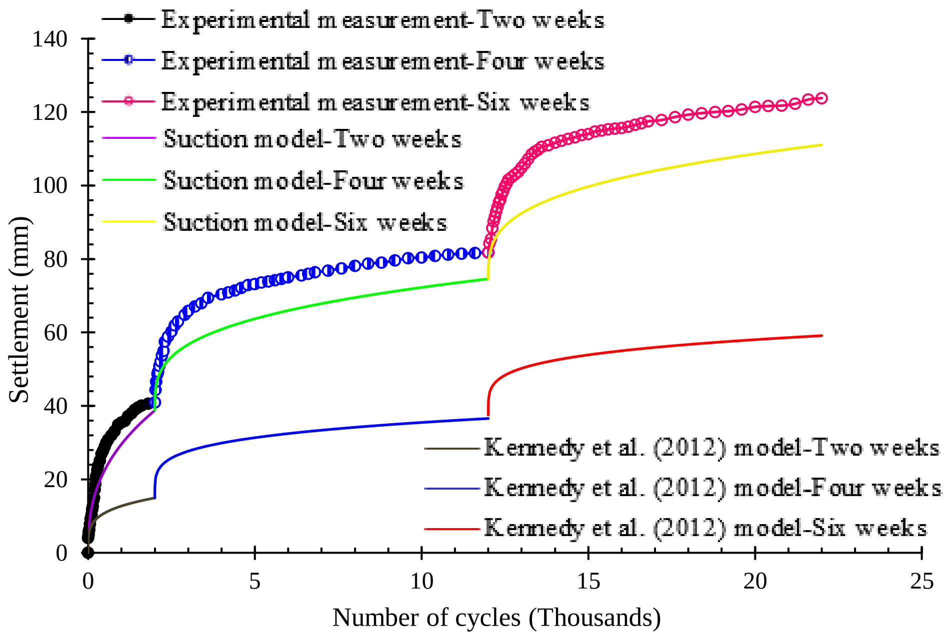
(c)

Figure 6.18 Comparison of track settlement models with the Kenndey et al. (2012) model, proposed model and experimental measurements (a) initial-dry, (b) 1st flooding-wet and (c) new surface layer-dry

During the recovery period, in the Experiment-1, Phase-III, the proposed model's track settlement predictions are excellent. The predicted settlements are approximately similar with experimental measurement is shown in Figure 6.19a. The subgrade is considered following wetting path as the suction decreases in the subgrade. Figures 6.19b and c present a comparison between experimental measurements and predicted track settlement in Experiment-2, Phase-II & III (during recovery period). The predicted settlement in Experiment-2, Phase-II, is approximately 5% lower after two and four weeks and 10% lower after six weeks (Figure 6.19b). However, in the Experiment-2, Phase-III, settlement was overestimated approximately 14% by the proposed model (Figure 6.19c). The Kennedy et al. (2012) model under predicts the track settlement in each Phases. Table 6.3 presents a comparison of the settlement difference between experimental measurement, Kennedy's model and the proposed model. The proposed model (based on suction) predicts the track settlement in the GRAFT more accurately in comparison with Kennedy model, particularly in the wet condition. However, further research is required as some of the parameters were not considered in this model, for instance, shear strength of soil.



(a)



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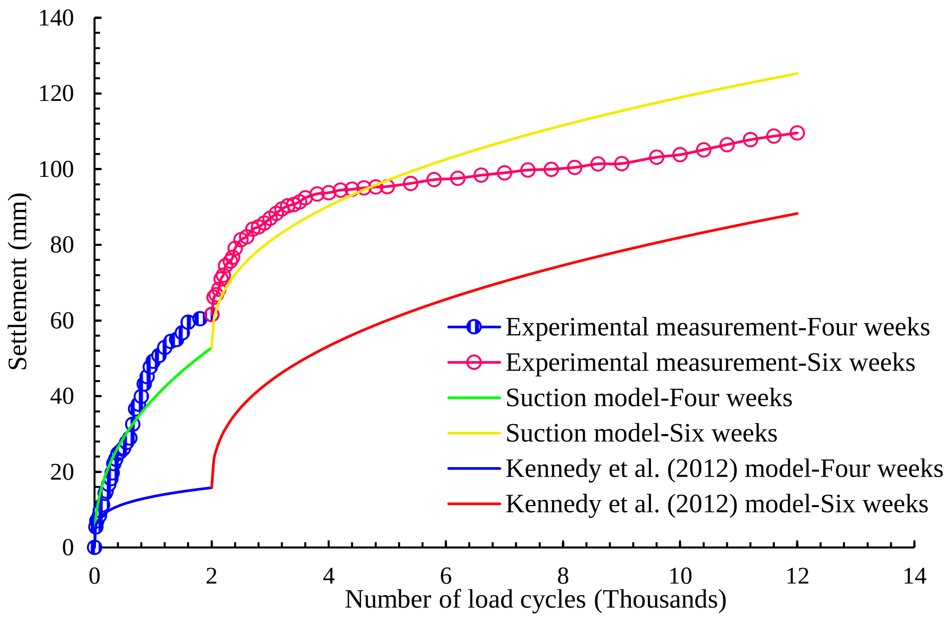

(c)

Figure 6.19 Track settlement comparison between experimental measurement and proposed model (a) Experiment-1, Phase-III, (b) Experiment-2, Phase-I and (c) Experiment-2, Phase-II

Table 6.3 Comparison with experimental measurements and model data

Experiments		a	b	No. of cycles (thousands)	Exp. settlement (max.) (mm)	Suction model (max.) (mm)	Kennedy et al. (2012) model (mm)
Exp-1, Phase-I		1.02	0.16	200	7.6	7.2	6.5
Exp-1, Phase-II		1.49	0.31	100	64.0	52.9	29.1
Exp-1, Phase-III	2 wks.	1.51	0.34	2	38.0	43.8	19.9
	4 wks.	1.48	0.30	(10+2)=12	73.80	72.7	39.8
	6 wks.	1.41	0.30	(12+10)=22	107.2	100.3	59.7
Exp-2, Phase-I	2 wks.	2.00	0.39	2	41.0	38.8	14.9
	4 wks.	2.26	0.30	(10+2)=12	81.8	74.6	37.5
	6 wks.	2.10	0.31	(12+10)=22	123.8	111.1	59.1
Exp-2, Phase-II	4 wks.	2.01	0.43	2	61.6	52.8	15.0
	6 wks.	1.66	0.41	(10+2)=12	109.6	125.3	88.3
Exp-2, Phase-III		1.36	0.24	300	47.2	28.1	10.4

6.4 Volumetric behaviour

Some soils experience volume change behaviour upon wetting, at natural moisture content soil can support heavy loads but with an increase of moisture content it faces a significant reduction in volume (Houston et al., 2002; Murthy, 2002). Generally, it is assumed that only sandy or silty soils show collapse but in recent studies it has been reported that compacted soil, in general, can show collapse (Tadepalli and Fredlund, 1991; Cox, 1978; Barden et al., 1973). Three double oedometer and two single oedometer tests were performed to investigate the volumetric and collapse behaviour at different conditions. In the previous section, the subgrade stiffness before and after flooding was discussed; this section explains the volumetric behaviour and response of the soil at wet condition.

6.4.1 Compression behaviour

In GRAFT, it was difficult to investigate subgrade volumetric behaviour; therefore, small-scale tests were carried out to investigate the subgrade compression behaviour. However, in this experiment only static load was applied up to 400kPa. The soil samples were collected from the GRAFT. The first experiment was at an initial condition where the moisture content was 10% and the matric suction 1300kPa. After 1st and 3rd flooding, samples were collected to do the second and third experiments. There were no tests performed after the 2nd flooding due to non- availability of the necessary equipment. Table 6.4 presents a summary of the results.

Table 6.4 List of oedometer test results

Initial state			Final state			Condition of test
Water content (%)	Void ratio, e_0	Degree of saturation (%)	Matric suction (kPa)	Water content (%)	Void ratio, e_f	
10.00	0.91	28.84	1300	8.27	0.81	Constant water content
	0.91	28.84		37.73	0.53	Wetted at 5kPa
	0.91	28.84		36.80	0.76	Wetted at 200kPa
25.00	0.80	83.50	150	23.37	0.55	Constant water content
	0.80	83.50		37.12	0.51	Wetted at 5kPa
	0.80	83.50		35.96	0.52	Wetted at 200kPa
30.00	0.80	90.96	90	26.88	0.53	Constant water content
	0.80	99.96		36.84	0.46	Wetted at 5kPa

Figures 6.20-22 present the double and single oedometer test obtained results. The samples' properties were at initial water contents of 10%, 25% and 30%; the initial void ratio was 0.91 for the first experiment, with the second and third experiments' void ratio being 0.80. Figure 6.23 compares the double oedometer results; it clearly shows the settlement difference between dry sample and wetted sample even for wetting at low stress. On entry of water whether at 5kPa or 200kPa, the soil volume changed significantly. However, an increase of initial moisture content decreased the settlement difference between constant water content and saturated sample, as initial moisture content was high and both samples were wet. The following section explains in detail about the collapse behaviour of the samples.

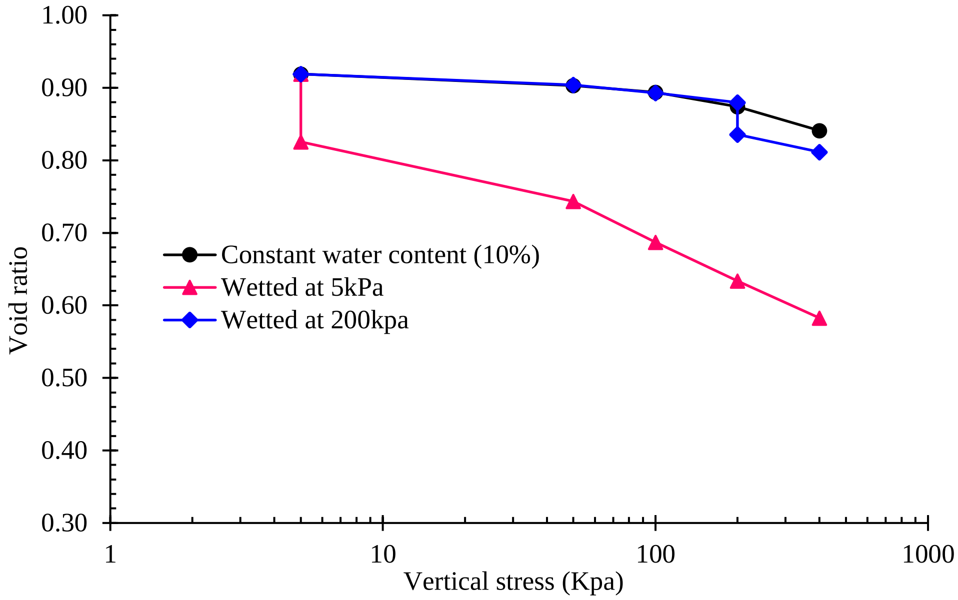

Figure 6.20 Double and single oedometer test at initial condition (10% moisture content)

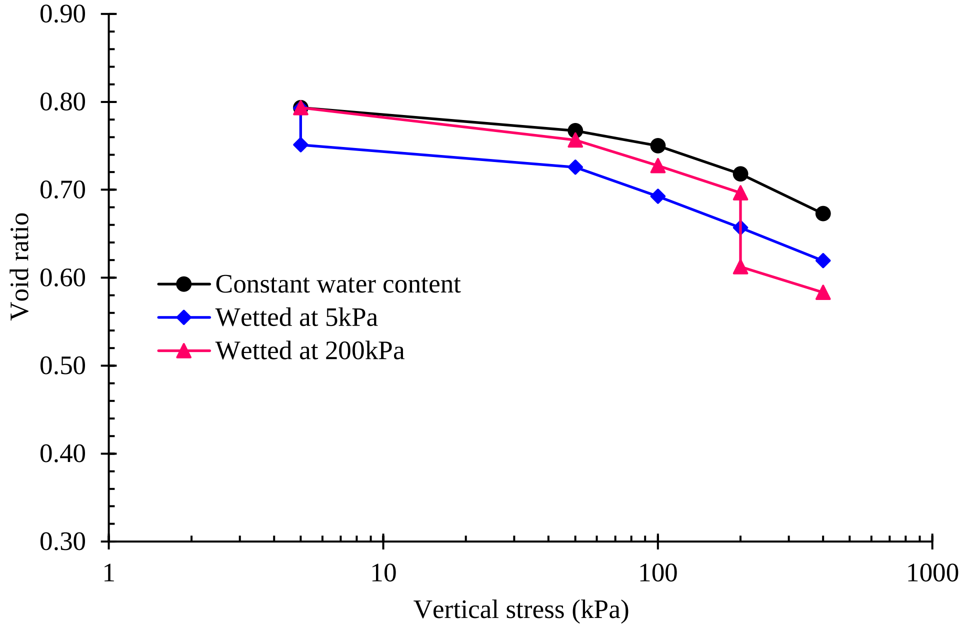

Figure 6.21 Double and single oedometer test at 25% moisture content

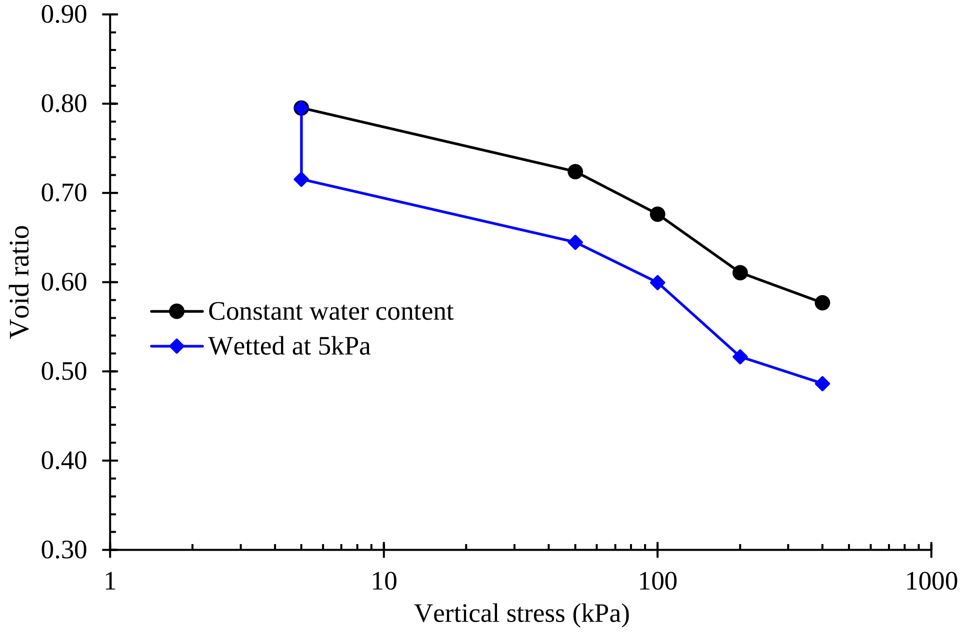

Figure 6.22 Double oedometer test at 30% moisture content

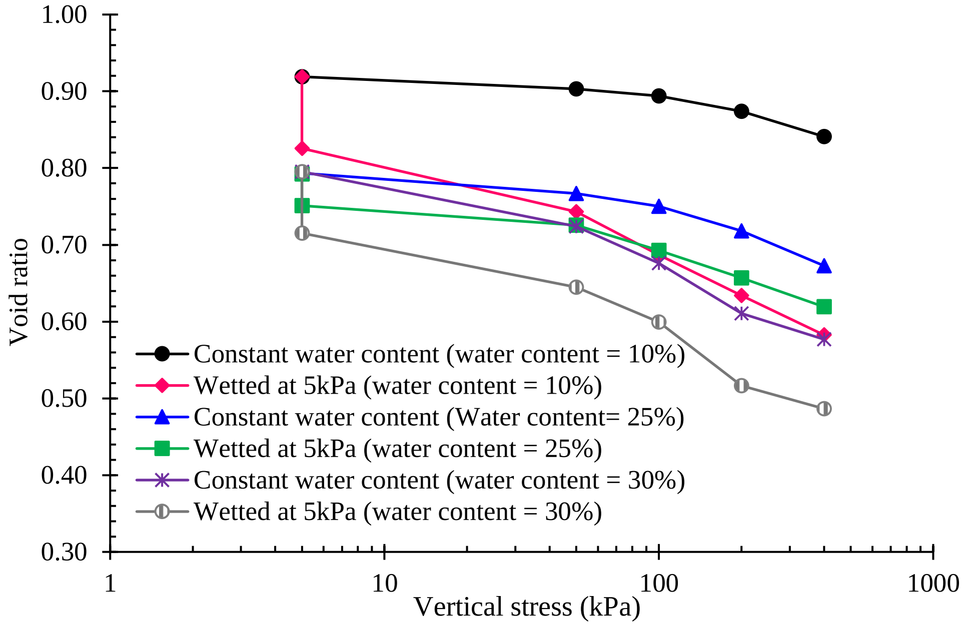

Figure 6.23 Comparison of double oedometer test at different subgrade condition

6.4.2 Collapse behaviour

Collapse is a function of the of the relative proportions of each component, including pre wetting moisture content or degree of saturation, initial void ratio, stress history of the materials, thickness of the collapsible strata and the amount of added load (Murthy, 2002; Lim and Miller, 2004). Delage et al. (2005) stated that collapse is a significant volume reduction when an unsaturated sample is wetted and under load. Lim and Miller (2004) reported that collapse settlement is a time dependent process which occurs from post construction increase of water content resulting from precipitation, capillary water from soil and from flooding.

Figure 6.24 presents the collapse behaviour of the subgrade soil for three different phases. The intent of this paragraph is to explain the behaviour of subgrade soil before and after flooding. Before flooding the subgrade was in an unsaturated condition but after flooding it became saturated and weak, particularly its surface layer. It was observed that the track experienced higher settlement due to flooding in GRAFT (discussed in previous chapters); the subgrade condition changes from unsaturated to saturated condition. In the first experiment (initial condition) collapse potential was almost 15%, where in other phases the collapse potential decreased. As in the first experiment, the sample water content was very low in the drier states; therefore, it shows very high collapse behaviour. Collapse potential increased by a decrease in the initial water content of the soil, increasing the total overburden pressure and decreasing of dry unit weight. Collapse potential decreased linearly with increased initial moisture content for a constant initial dry unit weight, as shown in Figure 6.25 (Fredlund and Gan, 1995; Lim and Miller, 2004).

Collapse of the soil generally happens upon wetting, due to a decrease of soil suction. Collapse behaviour is primarily related to the reduction of matric suction during inundation (Houston et al., 2002; Tadepalli and Fredlund, 1991). Tadepalli and Fredlund (1991) reported that the concept of effective stress is very useful for predicting the behaviour of saturated soil but it cannot explain the collapse behaviour of unsaturated soil during inundation; soils which collapse during inundation are initially unsaturated and consequently become saturated after inundation that lasts for a period of time.

In the 2nd and 3rd experiments, the initial water content increased and the matric suction decreased because of flooding; therefore, the subgrade soil shows lower collapse potential. The soil is wetter and more compressible; therefore, the bonding between particles became weak.

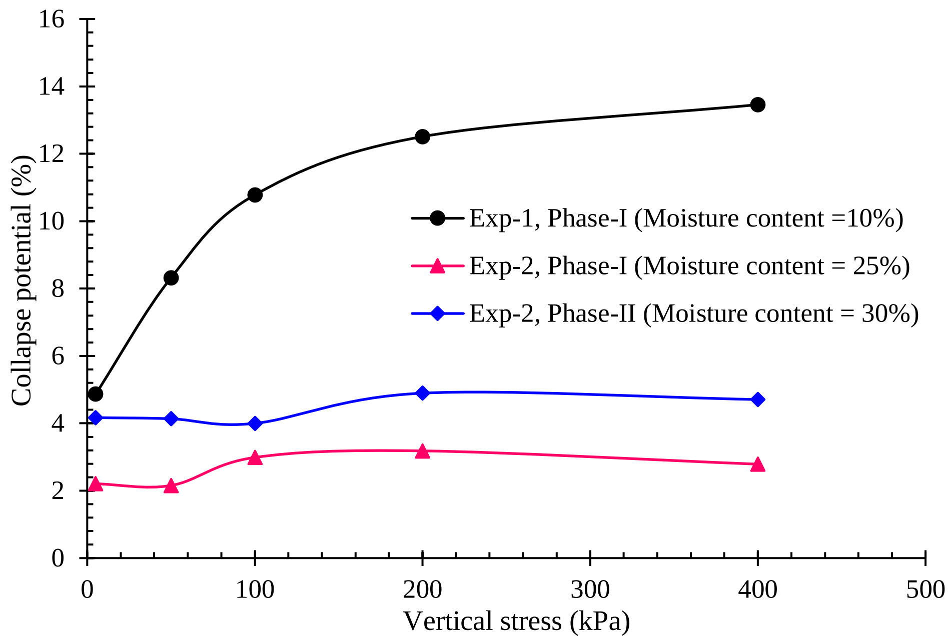

Figure 6.24 Collapse potential at different condition of subgrade

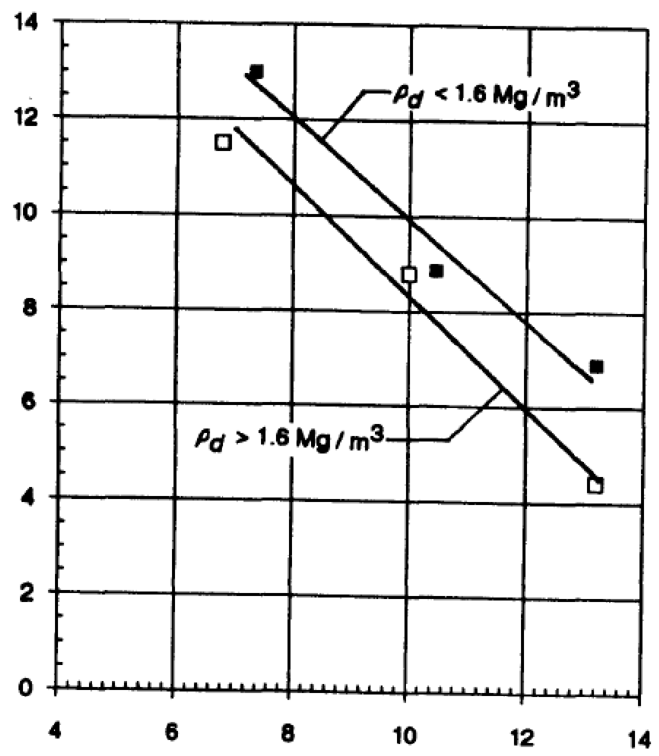

Figure 6.25 Effect of initial moisture content on the amount of collapse (Fredlund and Gan, 1995)

6.5 Concluding remarks

Track performance and maintenance are highly dependent on subgrade soil. The main role of the subgrade is to give adequate support to the entire structure but the performance reduces when water enters into the subgrade. It is not possible to stop water entering into the subgrade layer but it is possible to encourage it to drain away quickly. However, sometimes water, which has become entrapped between ballast, or water that remains near the structure, could change the substructure's properties. In addition, cyclic wetting and drying can significantly change subgrade properties. The soil-water interaction induced by wetting-drying cycles is complicated, as it involves the coupled effects of the changes in water content, suction, stress, deformation and shear strength (Ng and Tse, 2008; Zhan L et al., 2007). The results in this chapter have shown how the track performance is affected by cycles of wetting and drying.

The subgrade soil experienced cycles of wetting and drying, so the track performance varied. Track performance depends on which path it is following: a drying or wetting path. Soil water retention curves explain the behaviour of the wetting and drying effect on soil. Before the air-entry value, the soil behaviour is dominated by bulk water, therefore, during the recovery period the track performance showed little improvement. An important finding is that the suction hysteresis plays an important role on subgrade behaviour.

The influences of suction and water content are important factors that should be considered in subgrade performance assessment. Therefore, it can predict service conditions and required maintenance work. The entire track stiffness reduced after repeated flooding. Despite replacing its surface layer, the track showed poor performance due to low levels of soil suction in the entire subgrade soil. The result suggests that soil suction should be checked for the entire subgrade soil, not only for the upper portion. The relationship between matric suction and stiffness has shown that higher suction indicates higher stiffness and lower suction signifies subgrade that is less stiff. The suction increases the integrity of soil structure by, for example increasing rigidity of the soil skeleton.

The cyclic wetting and drying exerts a significant effect on subgrade soil behaviour. Therefore, despite a stiffer surface layer in the Experiment-2, Phase-III, the track settlement was high. Furthermore, the experiment with a subgrade soft bottom section has shown the effect of a rising water table, which can influence the entire subgrade's properties, and hence track performance. After extreme rainfall or repeated flooding, particular attention should be given to substructure, as in such circumstances the water content increases significantly.

A track settlement model is proposed based on soil suction hysteresis. The proposed model can predict track settlement more accurately in the GRAFT in different conditions of subgrade. Some of the models are only proposed based on applied load and where subgrade was considered in good condition (high stiffness and strength). These models are predict track settlement in normal condition of the track, where ballast settlement is the main factor. However, for long-term behaviour, subgrade plays an important role and the subgrade's behaviour not only depends on applied load but also environmental conditions.

From the oedometer test, the soil became more compressible and weak after repeated flooding. The test has shown volumetric changes between dry sample and wet sample under heavy load. In the dry state, soil showed little settlement but after wetting the settlement increased significantly. In particular, after the 3rd flooding, the volumetric behaviour of the soil at constant water content and wetted at 5kPa was almost the same, because of the low suction value and higher water content.

It is widely accepted that proper inspections, maintenance and repair of subgrade have been neglected over long period of time; analysis based on unsaturated soil mechanics is more relevant for an effective, economical and safe track design and maintenance (Selig and Waters, 1994; Li and Selig, 1995; Bonnet, 2005). This current investigation has shown the importance of soil suction and water content data, to any analysis of subgrade soil behaviour.

CHAPTER SEVEN-CONCLUSION

7.1 Conclusion

A series of full-scale tests was undertaken in the GRAFT to investigate the impact of flooding and during recovering period on railway track performance. The influence of moisture content, soil suction and the cyclic wetting and drying effect on track stiffness and settlement were studied in this research. Subgrade stiffness varied with changes of moisture content and suction; hence track performance varied. The major conclusions drawn from this research are as follows:

GRAFT: Repeated flooding

- The track experienced significant softening after flooding, resulting in the high track settlement. Initially, in the dry state (unsaturated condition), a half million cycles were applied at 3Hz without any problems. However, after flooding (saturated condition), only 2.5×10^5 loading cycles were applied and the loading frequency was reduced to 2Hz, the settlement increased significantly. The track required frequent maintenance (ballast tamping) to keep the track functional. An important finding of the research, if tamping of the track does not work, then it is a fault of the subgrade. Therefore, it cannot bring the track up to level due to the permanent settlement. In addition, frequent tamping can damage ballast properties. Furthermore, the large shear box test was conducted with dry and wet (7 days under water) ballast. The results did not show significant difference between dry and wet ballast. The investigation evidently showed that subgrade plays an important role for the track behaviour (i.e. maintenance work).

- Sand blanketing neither improved track performance during the wet period nor during the recovery period. However, sand blanketing stopped ballast penetrating into the soil, prevented slurry formation and blocked subgrade soil movement upwards into the ballast. The test with water inside the tank showed poor track performance in terms of a high settlememnt rate with the entire track quickly submereged. The main function of sand blanketing to drain water quickly from the track. But, if the drainge sytem blocked or insufficient design (prolonged flooding) then the sand blanketing causes significant problem on the track. The results showed that the track settlement was higher (with sand blanketing) than the the track without sand blanketing. However, a few weeks (i.e. four weeks) later (after drianed water), the track with a sand blanketing showed some improvement and showed quick recovery.

- During the recovery period, track settlement after two weeks of flooding (after drained water) was significantly high in comparison with the test conducted immediately after drained water. The track settlement reduced after four and six weeks but not significantly. The loading frequency during the recovery period was 2Hz.

- From visual inspection on removal of the ballast, a layer (approximately 50mm) of ballast penetrated into the subgrade soil and some ballast was contaminated. Foul ballast was one of the reasons for poor track performance.

Soil suction and subgrade modulus

- Repeated flooding resulted in a continual reduction in soil stiffness. After a third flooding, the stiffness was significantly low (compared with the dry state), as the suction declined because of the repeated flooding. It was found that the track stiffness reduced after flooding resulting from decreased soil suction. A relationship between matric suction and stiffness was developed which showed that high suction indicates higher stiffness and vice versa.

- The subgrade modulus was determined based on matric suction. The same moisture content can have two matric suction which is depend on the drying or wetting path. It is evidently showed that it is difficult to quantify the relationship between moisture content and subgrade modulus.

- A model is proposed in terms of track settlement, soil suction and number of applied cycles. These relationships fit the GRAFT data presented well in this thesis. The model can predcit track settlement in different conditions (unsaturated and saturated) of subgrade. It is important to investigate whether, the track is following wetting or drying path (due to cyclic wetting and drying path). The track behaviour significantly depends on, which path (drying/wetting) it is following.

Cyclic wetting and drying effect

- The effect of cyclic wetting and drying on subgrade behaviour was studied in this thesis. The tests were conducted by pressure plate and filter paper to determine the WRCs. The WRC explained the behaviour of subgrade soil at the wet and the dry condition. The soil suction is different at wetting and drying path (same moisture content has two different suction levels). Therefore, only moisture content cannot explain soil behaviour adequately. Before, the air- entry value, the soil behaviour is control by bulk water. Therefore, during the recovery period the track settlement was high as the suction level was under the air entry value.

- In the event of a flood, the upper layer of the subgrade is affected most; being sensitive to changes of water content and soil suction. However, over time the entire subgrade can be affected if the water stays in the track for a long period. In the experiments, it was observed that the wetting process was considerably faster than the drying process. Furthermore, repeated flooding altered the entire subgrade properties (i.e. mosture content, soil suction and the degree of saturation).

- Collapse behaviour was studied by double and single oedometer tests. In the GRAFT, it was difficult to identfy subgrade volumetric changes. The samples were collected directly from the GRAFT at different stages of the experiment. The collapse potential was high at the initial stage, when the soil transformed from dry state to wet state. At the later stage, collapse potential was not high as the moisture content of both samples was very high and the soil became weak and compressible. There were no significant differences between dry and wet samples.

- The assesment of subgrade soil should not be limited to the upper section only. An entire subgrade inspection should be done after flooding. It is important to have advanced knowledge about the subgrade soil behaviour at different condition (unsaturated and saturated), to be able to design an effective track. From a geotechnical point of view, to design a track yet involves traditional soil mechanics. It is essential to apply advanced soil mechanics to reduce maintenace costs and enhance track performance in the long-term.

7.1.1 Concluding comments

The research showed that track performance and maintenance are highly dependent on subgrade soil. The main function of the subgrade is to give adequate support to the entire structure, but the performance reduces upon water entering into the subgrade. It is not possible to stop water entering the subgrade soil but it can drain away quickly. However, cyclic wetting and drying can change subgrade properties significantly. For example, it changes in water content, soil suction, stiffness and strength. Therefore, it is essential to understand the effect of cyclic wetting and drying on subgrade behaviour. The research suggests that the subgrade soil assessment is essential after a flooding event; particularly repeated flooding events. The suction hysteresis should be considered for designing and maintenance work.

Track deterioration or track maintenance costs are all directly or indirectly related to the track's drainage system. All kinds of major substructure problems, particularly subgrade problems, are caused by water. Therefore, an effective track design needs an adequate drainage system. There is substantial information available about surface drainage, but little information available on subsurface drainage. To investigate the influence of traditional sand blankets two tests were conducted. The results show sand blankets could not improve track performance but could cure subgrade erosion problems. For the period of maintenance work, all the attention is given to ballast properties. Nonetheless, subgrade properties should be checked, especially after an extreme event, for example an over rainfall. A monitoring device (which can measure moisture content, soil suction and temperature) can be implemented in the track; especially in flood prone areas.

7.2 Recommendation for future research

The current experimental programme fulfils the research objectives and specifies a better understanding of subgrade soil. However, some pertinent issues regarding experimental techniques, results and existing knowledge in the literature are not explained. Therefore, the following areas are recommended for future research work:

- The experiments were conducted on a plain subgrade without embankment and only one week of flooding was considered in this research. The tests could be performed at different durations of flooding. In addition, different heights of flooding and velocity of water were not studied. The impact of flooding of different durations, levels and velocities are different and should therefore be studied.

- Within this research only track stiffness was measured. The shear strength test can be run in a suction controlled triaxial testing facility to obtain a better understanding of subgrade soil and the influence of suction upon it. The shear strength parameter should be considered in the proposed model. A suction control triaxial test is recommended to evaluate the data for the proposed model. Positive pore pressure was not measured in this research. There was a plan to measure positive pore pressure but it was not possible because of a lack of budget, implementation difficulty in the GRAFT and calibration problems.

- Only the filter paper technique was used to measure soil suction in the GRAFT. The filter paper test is a reliable but lengthy process. An alternative technique, employing a tensiometer, could be used to measure the soil suction, as it is a very quick process when compared to filter paper. Water retention curves were only shown for one cycle. In the pressure plate test, another ceramic plate could be added to get more cycles of wetting and drying.

- The relation was developed based on experimental data without suction hysteresis. Subgrade stiffness was determined test can be performed in a small-scale test with suction control. Therefore, a relation can be developed based on wetting and drying path.

- The final recommendation is for a numerical analysis of the flow behaviour of subgrade soil to be carried out.

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APPENDICES

Appendix A

Table A.1 Summary of subgrade properties of experiment-1 at Phase-I

Experimental phase	Sample Depth (mm)	No. of Sample	Moisture content, w (%)	Void ratio, e	Suction (kPa)		Degree of saturation, S_r (%)
					Matric	Total	
Experiment-1							
Phase I (Initial)	First Sleeper						
	100	1	10.49	0.91	1311.95	2058.69	30.43
		2	10.35	0.91	1300.79	2073.07	30.03
	200	1	13.08	0.91	1152.18	1806.49	37.95
		2	12.45	0.91	1107.06	1865.61	36.12
	300	1	14.29	0.91	1036.35	1696.43	41.46
		2	14.10	0.91	1023.48	1713.43	40.91
	400	1	16.13	0.91	957.81	1536.89	46.79
		2	15.28	0.91	903.17	1609.50	44.33
	500	1	17.07	0.91	869.23	1458.62	49.52
		2	16.67	0.91	844.45	1491.68	48.36
	Middle Sleeper						
	100	1	10.58	0.91	1292.46	2341.86	30.69
		2	9.51	0.91	1428.24	2377.25	27.59
	200	1	12.30	0.91	1223.09	1879.89	35.68
		2	11.25	0.91	1301.56	1982.06	32.64
	300	1	14.45	0.91	1037.71	1682.20	41.92
		2	14.26	0.91	1050.51	1699.11	41.37
	400	1	15.80	0.91	726.13	1564.87	45.84
		2	15.07	0.91	745.30	1627.72	43.72
	500	1	16.70	0.91	742.95	1489.19	48.45
		2	16.62	0.91	771.28	1495.84	48.22
	Third Sleeper						
	100	1	10.50	0.91	1192.00	2057.67	30.46
		2	10.39	0.91	1104.32	2068.95	30.14
	200	1	13.29	0.91	1092.27	1787.07	38.56
		2	13.10	0.91	1105.64	1804.63	38.00
	300	1	14.29	0.91	1023.48	1710.73	41.46
		2	14.13	0.91	1034.31	1696.43	40.99
	400	1	15.28	0.91	893.68	1609.50	44.33
		2	15.06	0.91	907.61	1484.21	43.69
	500	1	16.76	0.91	642.69	1628.59	48.62
		2	16.28	0.91	688.54	1524.26	47.23

Table A.2 Summary of subgrade properties of experiment-1 at Phase-II

Experimental phase	Sample Depth (mm)	No. of Sample	Moisture content, w (%)	Void ratio, e	Suction (kPa)		Degree of saturation, S_r (%)
					Matric	Total	
Experiment-1							
Phase-II (after flood)	Middle Sleeper						
	100	1	44.26	0.80	…	…	100.00
		2	42.92	0.80	…	…	100.00
	200	1	34.86	0.80	…	…	100.00
		2	32.21	0.80	9.30	87.98	100.00
	300	1	29.19	0.80	49.50	257.57	96.33
		2	28.04	0.80	139.96	412.51	92.53
	400	1	27.34	0.80	153.45	441.56	90.22
		2	27.06	0.80	427.48	585.15	89.30
	500	1	27.24	0.80	442.95	589.19	89.89
		2	26.08	0.80	471.28	605.84	86.06

Table A.3 Summary of results of experiment-1 at Phase-III

Experimental phase	Sample Depth (mm)	No. of Sample	Moisture content, w (%)	Void ratio, e	Suction (kPa)		Degree of saturation, S_r (%)
					Matric	Total	
				Experiment-1			
Phase III							
Two days				Middle Sleeper			
	100	1	37.78	0.80	...	...	100
		2	35.12	0.80	...	...	100
	200	1	32.32	0.80	39.67	223.11	100
		2	31.00	0.80	154.44	308.45	100
	300	1	29.83	0.80	265.83	627.75	98.44
		2	29.39	0.80	249.32	729.51	96.99
	400	1	27.04	0.80	423.91	717.14	89.23
		2	26.38	0.80	435.51	732.95	87.05
1 week				Middle Sleeper			
	100	1	36.12	0.80	...	...	100
		2	34.21	0.80	...	...	100
	200	1	32.74	0.80	5.71	250.16	100
		2	31.42	0.80	41.79	335.80	100
	300	1	26.61	0.80	100.35	454.80	87.81
		2	25.12	0.80	116.83	531.40	82.90
	400	1	25.30	0.80	199.17	578.41	83.49
	500	1	25.07	0.80	244.95	597.30	82.73
2 weeks				Middle Sleeper			
	100	1	35.06	0.81	...	...	100
		2	34.20	0.81	1.75	27.63	100
	200	1	31.00	0.80	57.98	335.80	100
		2	30.95	0.80	101.35	339.06	100
	300	1	27.84	0.80	215.47	544.80	91.87
	400	1	26.56	0.79	221.11	563.60	87.64
		2	25.00	0.79	177.66	607.77	83.54
	500	1	24.63	0.79	176.74	763.29	82.31
4 weeks				First Sleeper			
	100	1	27.68	0.80	103.17	226.97	91.34
		2	28.00	0.80	118.29	270.83	92.40
	200	1	29.15	0.80	167.03	457.45	96.20
		2	28.76	0.80	193.47	483.35	94.91
	300	1	28.68	0.80	238.60	488.68	94.64
		2	27.80	0.80	254.29	547.48	91.74
	400	1	30.18	0.80	178.59	389.48	99.59
		2	29.31	0.80	209.36	446.85	96.72
	500	1	30.12	0.80	18.77	73.07	99.4
		2	30.03	0.80	33.77	98.34	99.10

	Middle sleeper						
	100	1	27.78	0.80	183.61	205.13	91.67
		2	27.34	0.80	195.66	292.86	90.22
	200	1	28.32	0.80	213.27	250.16	93.46
		2	27.35	0.80	220.36	313.00	90.25
	300	1	28.87	0.80	214.45	476.04	95.27
		2	28.75	0.80	217.21	484.02	94.88
	400	1	29.68	0.80	186.33	422.40	97.94
		2	29.70	0.80	206.21	421.08	98.01
	500	1	28.32	0.80	83.78	245.53	93.46
		2	28.56	0.80	135.70	377.01	94.25
	Third sleeper						
	100	1	28.12	0.80	147.87	192.95	92.80
		2	27.86	0.80	163.55	233.40	91.94
	200	1	29.33	0.80	132.29	445.53	96.79
		2	28.90	0.80	140.26	474.04	95.37
	300	1	28.73	0.80	187.10	485.35	94.81
		2	27.45	0.80	208.56	571.00	90.59
	400	1	29.31	0.80	126.62	446.85	96.72
		2	29.15	0.80	135.70	457.45	96.20
	500	1	29.95	0.80	50.95	239.06	98.84
		2	30.18	0.80	67.96	289.48	99.59
6 weeks	First sleeper						
	100	1	27.64	0.80	183.61	558.23	91.21
		2	27.33	0.80	195.66	579.08	90.19
	200	1	26.88	0.80	213.27	609.47	88.70
		2	26.70	0.80	220.36	621.65	88.11
	300	1	26.85	0.80	214.45	611.50	88.61
		2	26.78	0.80	217.21	616.23	88.37
	400	1	27.57	0.80	186.33	562.93	90.98
		2	27.06	0.80	206.21	597.30	89.30
	500	1	29.74	0.80	103.71	418.44	98.14
		2	28.89	0.80	135.70	474.71	95.34
	Middle sleeper						
	100	1	29.34	0.80	118.71	444.87	96.82
		2	28.81	0.80	138.74	480.03	95.07
	200	1	29.91	0.80	97.37	407.24	98.70
		2	29.15	0.80	125.87	457.45	96.20
	300	1	28.38	0.80	155.12	508.67	93.65
		2	28.05	0.80	167.78	530.73	92.57
	400	1	29.66	0.80	106.70	423.72	97.88
		2	29.00	0.80	131.54	467.40	95.70
	500	1	30.21	0.80	86.22	387.51	99.69
		2	29.88	0.80	98.49	409.22	98.60
	Third sleeper						

100	1	28.57	0.80	147.87	496.00	94.28
	2	28.16	0.80	163.55	523.37	92.93
200	1	28.98	0.80	132.29	468.73	95.63
	2	28.77	0.80	140.26	482.69	94.94
300	1	27.55	0.80	187.10	564.28	90.92
	2	27.00	0.80	208.56	601.35	89.10
400	1	29.13	0.80	126.62	458.78	96.13
	2	28.89	0.80	135.70	474.71	95.34
500	1	29.68	0.80	105.95	422.40	97.94
	2	29.36	0.80	117.96	443.55	96.89

Appendix B

Table B.1 Summary of results of Experiment-2 at Phase-I (before flooding)

Experimental phase	Sample Depth (mm)	No. of Sample	Moisture content, w (%)	Void ratio, e	Suction (kPa)		Degree of saturation, S_r (%)
					Matric	Total	
Experiment-2							
Phase I (Before 2nd flooding	First Sleeper						
	100	1	29.86	0.80	62.25	410.53	98.54
		2	29.31	0.80	82.51	513.35	96.72
	200	1	29.43	0.80	180.26	573.30	97.12
		2	25.01	0.80	281.01	1880.36	82.53
	300	1	29.79	0.80	430.03	601.32	98.31
		2	25.24	0.80	399.18	947.44	83.29
	400	1	26.38	0.80	420.65	1463.44	87.05
		2	26.01	0.80	608.25	999.08	85.83
	Middle Sleeper						
	100	1	27.64	0.80	145.20	558.23	91.21
		2	27.33	0.80	157.04	579.08	90.19
	200	1	26.60	0.80	405.39	626.92	87.78
		2	25.92	0.80	337.24	985.63	85.54
	300	1	22.41	0.80	510.25	720.61	70.06
		2	21.23	0.80	570.76	728.73	73.95
	400	1	24.2	0.80	246.40	609.84	79.86
	500	1	25.07	0.80	255.33	719.74	82.75
	Third Sleeper						
	100	1	28.57	0.80	110.07	496.00	94.28
		2	28.16	0.80	125.49	523.37	92.93
	200	1	25.81	0.80	226.39	632.64	85.17
		2	24.80	0.80	287.54	794.63	81.84
	300	1	26.60	0.80	223.94	469.80	87.78
		2	26.70	0.80	385.45	885.49	88.11
	400	1	27.78	0.80	479.49	728.62	91.67
		2	26.31	0.80	222.21	338.31	86.82

Table B.2 Summary of results of Experiment-2, at Phase-I (after flooding)

Experimental phase	Sample Depth (mm)	No. of Sample	Moisture content, w (%)	Void ratio, e	Suction (kPa)		Degree of saturation, S_r (%)
					Matric	Total	
				Experiment-2			
Phase-I (After 2nd flooding				First Sleeper			
	100	1	36.97	0.79	...	...	100
		2	34.38	0.79	...	...	100
	200	1	27.98	0.79	122.80	441.85	93.50
		2	27.47	0.79	127.18	406.88	91.80
	300	1	27.26	0.79	176.77	459.45	91.10
		2	27.01	0.79	286.11	411.33	90.26
	400	1	26.33	0.79	182.77	414.39	87.99
		2	24.57	0.79	163.51	646.68	82.11
				Middle Sleeper			
	100	1	36.68	0.79	47.62	168.81	100
		2	37.35	0.79	55.04	101.70	100
	200	1	28.72	0.79	91.81	256.68	95.98
		2	27.92	0.79	134.41	311.57	93.30
	300	1	28.86	0.79	109.30	360.28	96.44
		2	28.72	0.79	52.29	170.80	95.98
	400	1	28.78	0.79	47.62	168.81	96.18
	500	1	29.26	0.79	55.04	101.70	97.78
				Third Sleeper			
	100	1	36.68	0.79	...	...	100
		2	37.35	0.79	...	...	100
	200	1	28.72	0.79	47.62	168.81	95.98
		2	27.92	0.79	55.04	101.70	93.30
	300	1	28.86	0.79	91.81	256.68	96.44
		2	28.72	0.79	134.41	311.57	95.98
	400	1	29.26	0.79	109.30	360.28	97.78
		2	28.78	0.79	52.29	170.80	96.18

Table B.3 Summary of results of Experiment-2, at Phase-II

Experimental phase	Sample Depth (mm)	No. of Sample	Moisture content, w (%)	Void ratio, e	Suction (kPa) Matric	Suction (kPa) Total	Degree of saturation, S_r (%)
Experiment-2							
Phase II	First Sleeper						
	100	1	35.22	0.79	...	...	100
		2	34.33	0.79	...	...	100
	200	1	27.78	0.79	111.40	527.15	92.83
		2	27.07	0.79	110.76	310.98	90.46
	300	1	31.08	0.79	145.54	295.19	103.86
		2	29.68	0.79	87.06	505.51	99.18
	400	1	30.43	0.79	76.73	22.99	100
		2	29.68	0.79	...	...	99.18
	500	1	29.11	0.79	83.67	531.32	97.28
		2	28.72	0.79	110.14	198.46	95.98
	600	1	29.43	0.79	195.91	290.37	98.35
		2	30.10	0.79	...	...	100
	Middle Sleeper						
	100	1	34.68	0.79	...	...	100
		2	33.59	0.79	...	...	100
	200	1	30.33	0.79	24.73	40.12	100
		2	29.22	0.79	16.52	60.10	97.65
	300	1	28.75	0.79	91.52	184.72	96.08
		2	28.56	0.79	115.66	180.65	95.44
	400	1	28.90	0.79	...	...	96.58
		2	28.02	0.79	34.39	61.46	93.64
	500	1	30.56	0.79	43.86	56.04	100
		2	29.84	0.79	23.08	57.76	99.72
	600	1	29.87	0.79	...	...	99.82
		2	29.81	0.79	...	...	99.62
	Third Sleeper						
	100	1	36.45	0.79	...	...	100
		2	35.05	0.79	...	...	100
	200	1	27.63	0.79	198.69	260.17	92.33
		2	27.22	0.79	85.66	201.00	90.96
	300	1	29.09	0.79	48.44	199.85	97.21
		2	28.60	0.79	110.45	234.08	95.57
	400	1	30.17	0.79	79.02	241.73	100
	500	1	29.07	0.79	79.91	153.83	97.15
		2	28.30	0.79	...	...	94.57
	600	1	29.78	0.79	59.67	197.69	99.52
		2	28.21	0.79	...	...	94.27
	700	1	31.27	0.79	...	...	100
		2	30.73	0.79	...	...	100

Table B.4 Summary of results of Experiment-2, at Phase-III

Experimental phase	Sample Depth (mm)	No. of Sample	Moisture content, w (%)	Void ratio, e	Suction (kPa)		Degree of saturation, S_r (%)
					Matric	Total	
Experiment-2							
Phase III	First Sleeper						
	100	1	27.19	0.81	104.06	303.78	88.62
		2	27.00	0.81	92.27	282.16	88.00
	Middle Sleeper						
	100	1	26.34	0.81	115.97	374.79	85.85
		2	24.53	0.81	97.49	217.82	79.95
	Third Sleeper						
	100	1	29.04	0.81	95.02	237.47	94.65
		2	28.59	0.81	88.63	358.86	93.18
After 8 weeks	First sleeper						
	100	1	12.88	0.81	233.57	980.76	64.79
		2	13.58	0.81	392.36	1834.39	63.82
	200	1	20.59	0.81	404.11	1860.43	67.11
		2	22.78	0.81	665.02	1245.73	74.25
	300	1	20.78	0.81	329.03	1361.56	67.73
		2	22.30	0.81	176.27	1132.93	72.68
	400	1	22.34	0.81	399.90	1052.54	72.81
		2	21.86	0.81	420.69	1087.52	71.25
	500	1	23.26	0.81	360.65	986.45	75.81
		2	22.28	0.81	402.48	1056.89	72.62
	600	1	23.28	0.81	359.81	985.03	75.88
		2	22.24	0.81	404.21	1059.80	72.49
	700	1	23.91	0.81	333.39	940.51	77.93
		2	23.68	0.81	342.99	956.70	77.18
	Middle sleeper						
	100	1	12.19	0.81	664.97	1468.04	47.62
		2	12.65	0.81	737.45	1396.30	42.99
	200	1	22.79	0.81	367.32	363.09	74.28
		2	22.04	0.81	409.06	823.10	71.83
	300	1	23.33	0.81	316.88	521.06	76.04
		2	22.56	0.81	...	...	73.53
	400	1	23.59	0.81	341.15	584.67	76.89
		2	23.46	0.81	352.22	1000.71	76.46
	500	1	23.26	0.81	566.00	936.51	75.81
		2	23.19	0.81	406.37	991.44	75.58
	600	1	23.90	0.81	233.08	550.71	77.90
		2	23.04	0.81	...	...	75.09
	700	1	23.55	0.81	348.57	814.30	76.76
		2	23.12	0.81	...	...	75.35

Third sleeper						
100	1	14.21	0.81	430.46	552.27	56.09
	2	15.14	0.81	...	...	49.35
200	1	23.86	0.81	570.93	600.78	77.77
	2	23.11	0.81	...	...	75.32
300	1	22.00	0.81	463.91	643.16	71.70
	2	21.93	0.81	335.56	514.03	71.48
400	1	22.06	0.81	455.45	517.27	71.90
	2	21.65	0.81	446.91	708.86	70.56
500	1	22.15	0.81	200.81	468.54	72.19
	2	21.26	0.81	346.14	653.05	69.29
600	1	22.01	0.81	370.26	574.85	71.74
	2	21.79	0.81	...	...	71.02
700	1	23.44	0.81	439.07	554.14	76.40
	2	23.14	0.81	...	...	75.42